THE CONTENTS OF THIS SECTION ARE
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INITIAL KH DATE 8/10/04

Engineering Design File

TAN-607A High Bay Floor Loading Evaluation

Portage Project No.: 2073.00
Project Title: PM-2A Remediation Phase 1

Portage ENVIRONMENTAL

ENGINEERING DESIGN FILE

1. Portage Project No.: 2073.00 2. Project/Task: PM-2A Remediation Phase 1

3. Subtask: TAN-607A High Bay Floor Loading Evaluation

4. Title: TAN-607A High Bay Floor Loading Evaluation

5. Summary:

This engineering design file evaluates the floor loading capabilities of the TAN- 607A High Bay to support placement of the PM-2A tanks and associated radiological shielding for treatment of the tank contents prior to disposal at the Idaho Comprehensive Environmental Response, Compensation, and Liability Act disposal facility.

6. Distribution: (Portage Environmental, Inc.)

Lisa Aldrich, PEI Document Control (Original)
Brady J. Orchard, P.E.
Nathan M. Wheldon, P.E.
Jeff A. Towers

7. Review (R) and Approval (A) Signatures:
(Identify minimum reviews and approvals. Additional reviews/approvals may be added.)

	R/A	Printed Name/ Organization	Signature	Date
Author	A	Jeff A. Towers		6-21-04
Independent Review	R	Nathan M. Wheldon, P.E.		6-21-04
Project Manager	R/A	Brady J. Orchard, P.E.		06/21/04

PROFESSIONAL ENGINEER
REGISTERED
10393
06/21/04
STATE OF IDAHO
BRADY J. ORCHARD

CONTENTS

1. INTRODUCTION AND PURPOSE

This engineering design file (EDF) evaluates the floor loading capabilities of the TAN-607A High Bay for placement of the PM-2A tanks and the necessary radiological shielding to allow continuous occupational occupancy of the High Bay as stated in 10 CFR 835.1002. Shielding requirements for the PM-2A tanks are described in Portage Environmental, Inc. (Portage) EDF, PEI-EDF-1005.

2. BACKGROUND

During Phase 1 remediation of the PM-2A tanks, the tanks will be excavated and transported to the TAN-607A High Bay for storage. Because the tanks contain sludge contaminated with radionuclides, shielding will have to be placed in the TAN-607A High Bay to prevent personnel exposure and allow unrestricted access for normal occupational occupancy while the tanks are in storage. The floor loading from the combination of the tanks, support structures, and associated shielding is expected to be significant. To prevent any potential damage to the High Bay floor, a structural engineering evaluation of the floor loading restrictions was performed by Eclipse Engineering, Inc. (Eclipse), under contract to Portage. The floor loading analyses are included in this EDF as Attachment 1.

3. ANALYSIS RESULTS

Eclipse performed the structural analysis of the TAN-607A High Bay floor based on the original structural drawings provided by Bechtel BWXT Idaho, LLC. In areas of the floor where additional reinforcing was done for specific project requirements since initial construction, no credit was taken for reinforcing that was not documented on as-built drawings. The Eclipse analysis divided the High Bay floor into seven sub-areas with specific floor-loading capacities. These sub-areas are delineated on Portage Drawing P-FFA/CO-PM2A-003 (Attachment 2).

Sub-Area 1 – 500 lb/ft^2

Sub-Area 2 – 2,615 lb/ft^2, except in the assembly pit area, which is limited to 285 lb/ft^2

Sub-Area 3 – 500 lb/ft^2

Sub-Area 4 – 1,542 lb/ft^2

Sub-Area 5 – 1,490 lb/ft^2

Sub-Area 6 – 500 lb/ft^2

Sub-Area 7 – 500 lb/ft^2, except the strip supporting the railroad tracks, which can support 1,895 lb/ft^2.

Eclipse evaluated the assembly pit in Sub-Area 2 and recommended a structural cover that would allow placement of the PM-2A tanks on Sub-Areas 2 and 4 without damaging the floor. The structural cover consists of tube steel planks (HSS 8 in. wide by 2 in. high by 20 ft long, 0.188-in. wall thickness) placed across the assembly pit area (57 ft 6 in. long by 4 ft wide) to transfer the load to the higher strength areas of Sub-Area 2. The evaluation of the area used a 205,000-lb total load on a six-axle transporter, with concrete shielding installed 12 ft from the planking centerline for personnel exposure protection. The tank transporter tires exerted 6,960 psf on the steel planking and the shielding added 870 psf per lineal foot approximately 1 ft away from the steel planking with the combination resulting in permissible floor loading. The analysis was performed using less axles (six versus 12) on the trailer to ensure adequate strength for the additional weight (approximately 100,000 lb) from anticipated grouting in Phase 2 that the transporter and floor will have to carry safely when the tanks are removed from the TAN-607A High Bay. Details are provided in the assembly pit cover design in Attachment 1.

For areas where the floor-loading capacity is less than the force applied by a point load like a tire, factors such as slab thickness, slab design, and overall floor loading must be examined to determine what point load can be safely applied to the floor. Concrete floors are designed to distribute loads so that an individual point on the floor never actually carries the entire load. As an example, Sub-Area 4 has a floor rating of 1,542 psf, but can safely support movement of a PM-2A tank on the transporter having an axle load of 34,167 psf (205,000-lb load, six axles) because of the floor design and lack of other additional floor loading (details in Attachment 1).

4. CONCLUSIONS

The TAN-607A High Bay floor will require a structural cover over the assembly pit area to safely allow anticipated floor loading from the PM-2A tanks and associated shielding. The floors outside the assembly pit will not require any additional strengthening or modifications to safely support floor loadings from transport and placement of the tanks and associated shielding in the High Bay. Additionally, the floor loadings and cribbing equipment in Sub-Areas 2 and 4 have been checked with the anticipated extra weight (100,000 lb) that grouting may add during Phase 2 activities and determined adequate. In the event that future operations require capacity floor loadings in localized areas of the High Bay, additional structural covers can be utilized to safely disperse the load.

5. REFERENCES

10 CFR 835.1002, 2004, "Facility Design and Modifications," *Code of Federal Regulations*, Office of the Federal Register, January 1, 2004.

PEI-EDF-1005, 2004, "PM-2A Tank Shielding Requirements using MicroShield v. 6.02," Rev. 0, May 2004.

Attachment 1

Assembly Pit Cover Design

Eclipse Engineering, Inc.
consulting engineers

April 30, 2004

Mr. Jeff Towers
Portage Environmental, Inc.
1075 South Utah, Suite 200
Idaho Falls, ID 83402

Re: Assembly Pit Cover Design
Building TAN 607A, High Bay Assembly Shop
Idaho National Engineering & Environmental Laboratory
Idaho Falls, Idaho

Jeff,

As requested, I have designed the cover for the Assembly Pit of the above noted building. The adjacent floor and foundation is a system of cast-in-place concrete slabs, grade beams and drilled concrete piers. The Assembly Pit, located within the area known as the <u>HIGH BAY ASSEMBLY SHOP</u>, is approximately 57'-6" long x 4'-0" wide. Reference our floor analysis report dated March 11, 2004. Also reference the original structural drawings produced by The Ralph M. Parsons Company, dated 8/3/56.

The owner wishes to store large tanks in the <u>HIGH BAY ASSEMBLY SHOP</u>. The tanks shall be transported into the building by entering through a door on the west side. The transporter will back the tanks along the existing railroad tracks to the east side of the Shop. The tanks will be positioned end-to-end along the tracks from the east side to the west door of the Shop (approx. 3 tanks).

In order to safely move or store the tanks in the Shop, a structural cover shall be constructed over the abandon Assembly Pit. This cover shall be HSS8x2x3/16x20'-0" tube steel planks that span across the 4'-0" width of the pit as shown on the attached Details A and B, sheets 1 and 2, respectively.

The planks shall support the total weight of the tank and the transporter, as well as the saddle & saddle support, which have a total weight of 205,000 lbs. Reference the attached information from 'Duratek', sheet 3. The transporter has 6 axles, so each axle is assumed to support 34,167 lbs. Each 10'-0" axle has 8 wheels (4 on each end of the axle). So each set of 4 wheels supports one-half of the axle load or

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17,083 lbs. The wheels exert 6,960 psf of pressure on the planks, which are adequate to support this pressure (reference the attached calculation on the attached sheet 4).

Regarding the remainder of the concrete floor in the Shop area, although the 6,960 psf of pressure is more than the 1500 psf allowed per my report dated March 11, 2004, the concrete is capable of dispursing the concentrated wheel load. The 6,960 psf wheel load only makes contact with about 10% of the floor at any given instant. And the other 90% of the floor is not loaded at all. So, what becomes important is the effective loaded area.

The bending and shear stresses in the concrete floor resulting from the concentrated load will be dispersed through the concrete slab over a width equal to about 8 times the slab thickness. Since the slab is 10-inches thick (minimum), the effective width of the loaded area is either 80-inches or the center-to-center spacing of the axles. Reference the attached calculation, sheet 5, which justifies the concrete floor to support the transporter. The steel plank cover over the Assembly area is not conjoined and therefore can not dispurse the load in the same manner as the concrete. Therefore, the planks shall be designed for the full 6,960 psf of load.

Also, the floor is adequate to support concrete shielding block walls located on either side of the tanks. It is my understanding that the shielding blocks are 20-inch thick concrete blocks that are stacked 6'-0" high. Assuming a concrete density of 145 pcf, the load on the concrete floor from the walls would be 870 psf, which is less than the load rating on the floor.

We have designed only the Assembly Pit Cover for the <u>HIGH BAY ASSEMBLY SHOP</u> as described in this letter. With the exception of our floor analysis report dated March 11, 2004, we hold no responsibility for any other element or the integrity of the structure as a whole. Please call with any specific questions.

Sincerely,

Eclipse Engineering, Inc.

PROFESSIONAL ENGINEER
REGISTERED
9081
4/3/04
STATE OF IDAHO
TROY LEISTIKO

Troy Leistiko, P.E.
Project Engineer

Attachments: Details A and B, 'Duratek' loading information, calculations.

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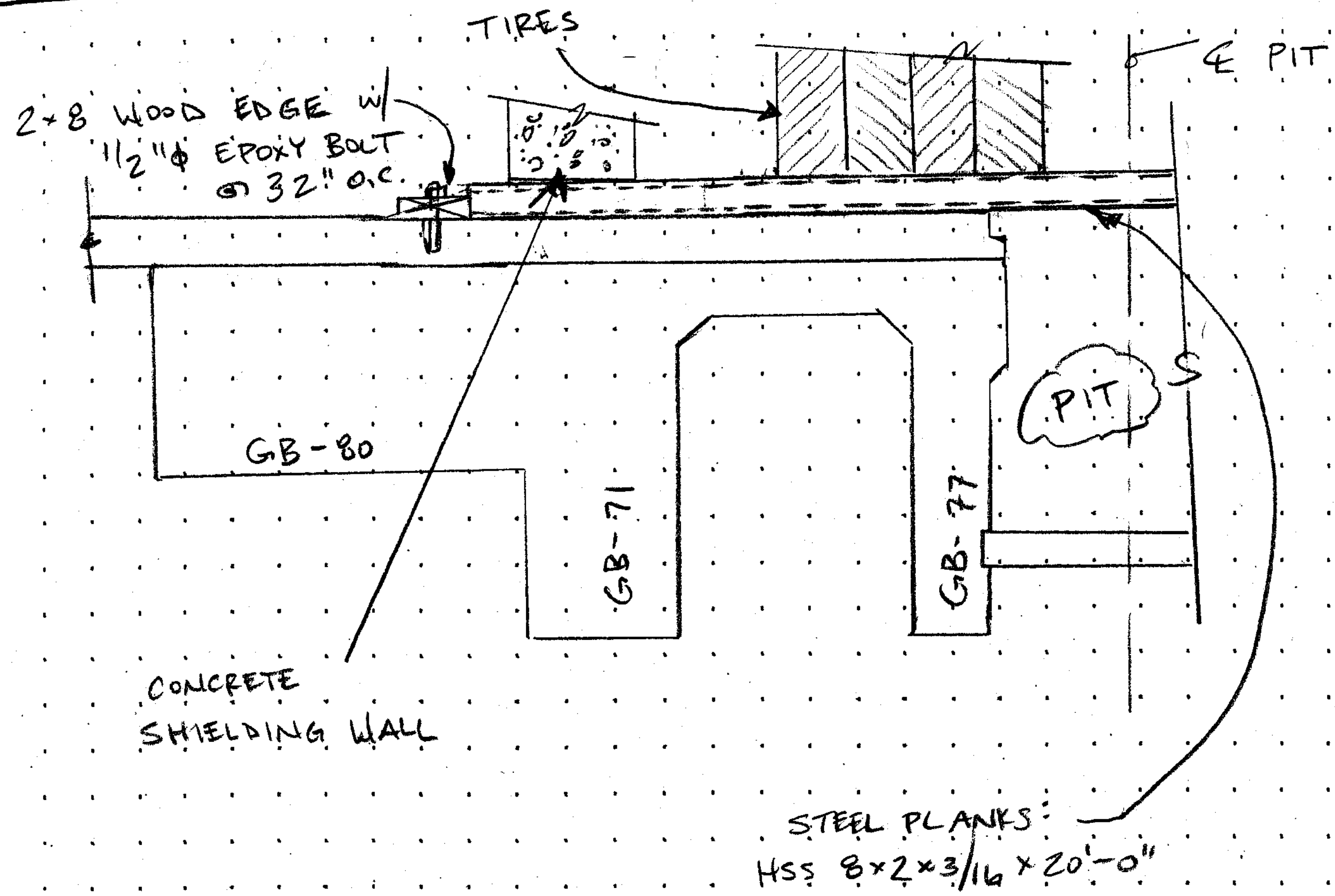
TAN BLDG 607A

TRANSPORTER + TANK SUPPORT

TROY

4/30/04



A PIT SECTION
SCALE: 3/8"=1'-0"

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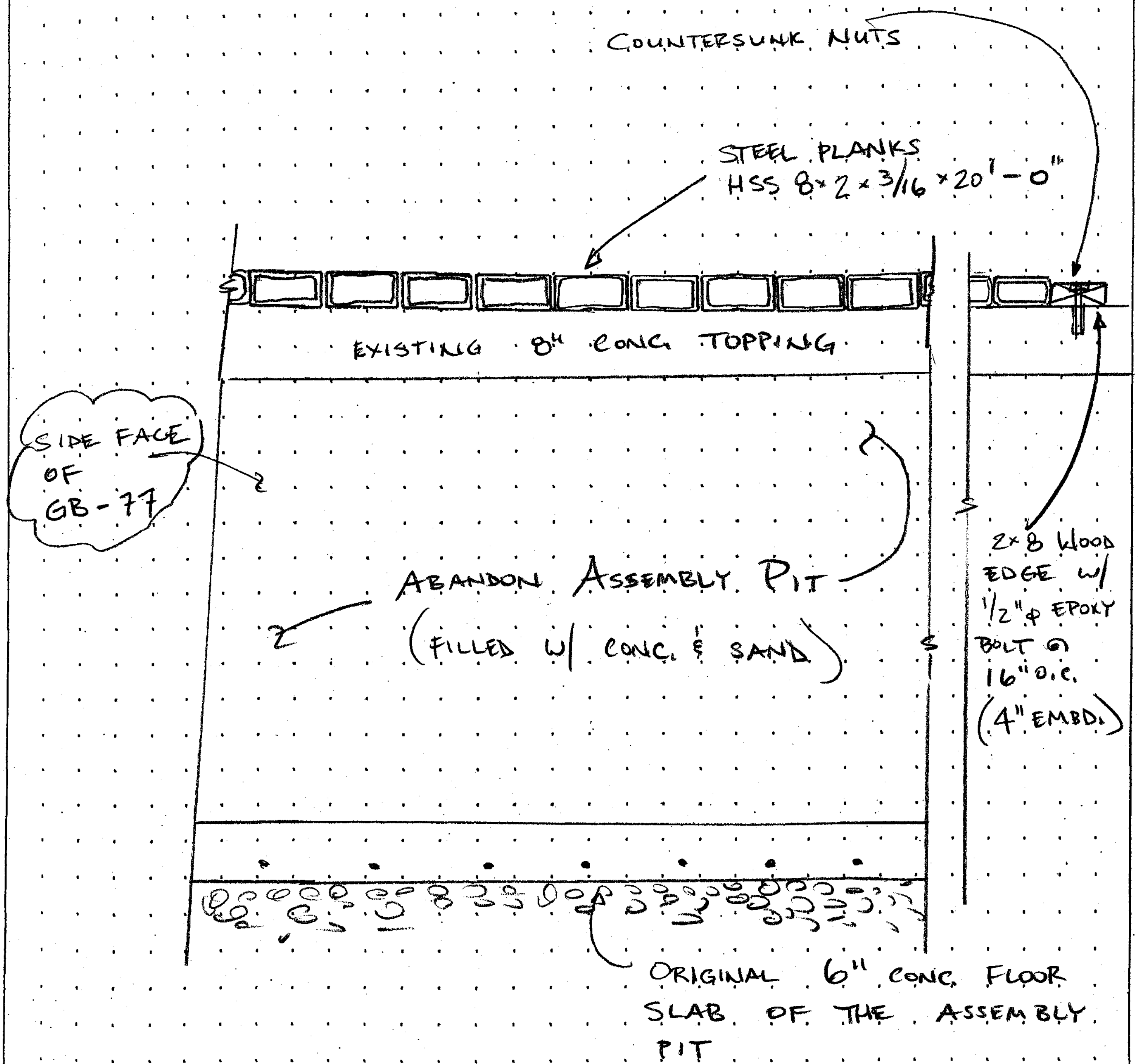

B PIT COVER SECTION

SCALE: 3/4"=1'-0"

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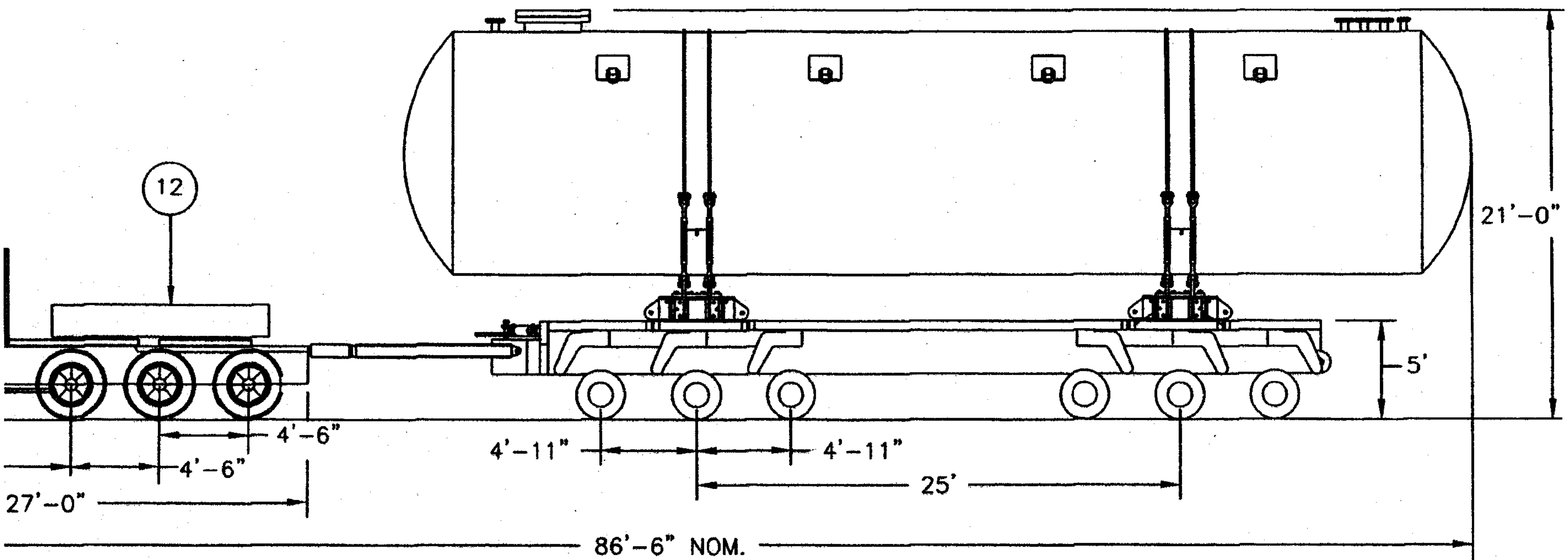
3/5



PRIME MOVER DATA

)NT	WEIGHT = 15,000 LBS	1 AXLES
	TIRES/AXLE	2
	TIRES CONTACT AREA	5.55 SQ.FT
	PSF	2,702
ACK	WEIGHT = 26,500 LBS	3 AXLES
	TIRES/AXLE	4
	TIRES CONTACT AREA	10.03 SQ.FT
	PSF	2,629

WT. UNITS

TRUCK	24,000 LB
COUNTERWEIGHT	17,500 LB
TOTAL WT.	41,500 LB

TRANSPORTER DATA

No. OF AXLES	6 AXLES
TIRES/AXLE	8
TIRES CONTACT AREA	88.36 SQ.IN.
TOTAL CONTACR AREA	4241.28 SQ. IN.
PSI	48.33
PSF	6,960

WT. UNITS

TRANSPORTER	40,000 LB
TANK	125,000 LB
SADDLE SUPPORT/SADDLE	40,000 LB
TOTAL LOAD	205,000 LB

☒ PROPRIETARY

☐ NON-PROPRIETARY

FSCM No. 54643

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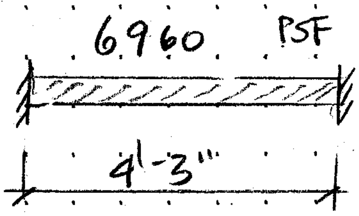
PIT COVER



ASSUME A500 TUBE
$F_y = 46$ KSI
$F_b = 0.75\ F_y$
$F_v = 0.40\ F_y$
$E = 29E6$ PSI
ANALYZE / DESIGN W/ ASD

$V = 14.79$ K/FT

$M = 10.48$ K'/FT

$A_{WEB,\ REQUIRED} = 0.80$ IN²/FT.

$S_{y,\ REQUIRED} = 3.64$ IN³/FT.

USE HSS 8×2×3/16 PLANKS

($A_{WEB} = 1.125$ IN²/FT)
($S_y = 3.78$ IN³/FT)
($I_y = 3.78$ IN⁴/FT)

CHECK DEFLECTION, $\Delta = \dfrac{w l^4}{384 EI} = 0.093''$, OK $L/547$

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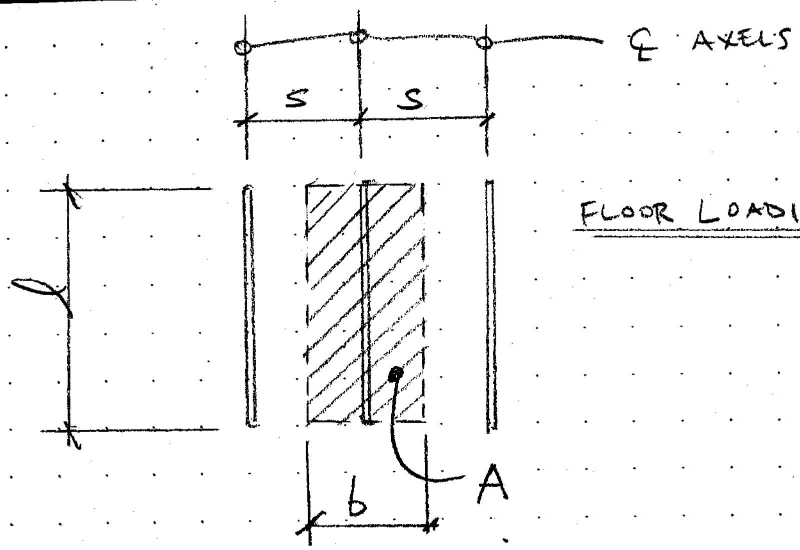
TAN BLDG 607A

TRANSPORTER & TANK SUPPORT

DATE: 4/30/04

DESIGN BY: TROY



FLOOR LOADING DIAGRAM
NO SCALE

l = EFFECTIVE LOADED LENGTH (OUT-TO-OUT DIMENSIONS OF TIRES)

b = EFFECTIVE LOADED WIDTH, THE LESSER OF:
8t OR S, WHERE...
t = SLAB THICKNESS
S = CENTER-TO-CENTER SPACING OF AXELS

A = EFFECTIVE LOADED AREA $= l \cdot b$

P = LOAD PER AXEL

w = EFFECTIVE LOAD PER SQUARE FOOT $= \dfrac{P}{A}$

EXAMPLE

P = 205,000 LB ÷ 6 AXELS = 34,167 LB.

l = 10'-0", b = 4'-11" ⟶ A = 49.17 FT.^{2}

w = 695 PSF > 1500 PSF <u>OK</u>

(END OF EXAMPLE)

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To:	Jeff Towers Portage Environmental – Idaho Falls	**From:**	Troy Leistiko
Fax:	(208) 523-8860	**Pages:**	~~8~~ 10 (including this cover)
Phone:	(208) 227-1393 – Direct Line to Jeff T.	**Date:**	5/12/04
Re:	TAN Bldg. 607A – Assembly Pit Cover	**CC:**	none

☐ **Urgent** x **For Review** ☐ **Please Comment** ☐ **Please Reply** x **Please Recycle**

• **Comments:**

Jeff,

Attached is my letter providing proof that the existing slab, when supporting a point load, shall be analyzed as a strip that is up to 8 times the slab thickness.

Please call with any questions.

Best Regards,

Troy

Eclipse Engineering, Inc.
consulting engineers

May 11, 2004

Mr. Jeff Towers
Portage Environmental, Inc.
1075 South Utah, Suite 200
Idaho Falls, ID 83402

Re: Assembly Pit Cover Design
Building TAN 607A, High Bay Assembly Shop
Idaho National Engineering & Environmental Laboratory
Idaho Falls, Idaho

Jeff,

As requested, this letter serves as a response to your request that we provide some sort of background or research to our assumption that the bending and shear stresses in the concrete floor resulting from the concentrated load will be dispersed through the concrete slab over a width equal to about 8 times the slab thickness. Reference our design letter dated April 30, 2004.

I have attached further calculations and excerpts from three references that I fell will convince you and the owner that our design assumptions are conservative.

Please call with any specific questions.

Sincerely,

Eclipse Engineering, Inc.

Troy Leistiko, P.E.
Project Engineer

Attachments: excerpts from ACI-318, Design of Concrete Structures (Nilson), and Reinforced Concrete Fundamentals (Ferguson, Breen, Jirsa)

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TAN BLDG 607A

TRANSPORTER + TANK SUPPORT

1/1

DATE: 5/11/04

DESIGN BY: TROY

- FROM ACI 318 8.10.2,
  EFFECTIVE FLANGE THICKNESS FOR T-BEAMS
  = 8 TIMES THE SLAB THICKNESS

- FROM ARTHUR H. NILSON,

$$P = 2\pi (M + M')$$

$$\phi P_n = 2\pi \left(8.69 \tfrac{k'}{FT.} + 8.69 \tfrac{k'}{FT.}\right) \doteq \underline{\underline{109.2\,k}}$$

$$109.2k >> 17.1\,k \ (\tfrac{1}{2} \text{ ASSUMED AXLE LOAD})$$

- FROM JOHANSEN,

EFFECTIVE WIDTH = $2 \times \text{SPAN} \cdot \sqrt{\mu}$,
WHERE μ = RATIO OF PERPENDICULAR TOP STEEL TO LONGITUDINAL BOTTOM STEEL

SPAN = 5'-7 1/4" (SECTION B/S115 OF PARSONS DRWGS.)

$$\mu = 0.44 = \frac{0.233 \text{ IN.}^2/\text{FT}}{0.528 \text{ IN.}^2/\text{FT.}} \quad \frac{(\#5 @ 16")}{(\#6 @ 10")}$$

$\therefore$ EFFECTIVE WIDTH = $2 \times 5.60' \times \sqrt{0.44} = 7.43$ FEET

OR 8.92 × SLAB THICKNESS WHICH EXCEEDS OUR ASSUMPTIONS

CODE

8.10 — T-beam construction

8.10.1 — In T-beam construction, the flange and web shall be built integrally or otherwise effectively bonded together.

8.10.2 — Width of slab effective as a T-beam flange shall not exceed one-quarter of the span length of the beam, and the effective overhanging flange width on each side of the web shall not exceed:

(a) eight times the slab thickness;

(b) one-half the clear distance to the next web.

8.10.3 — For beams with a slab on one side only, the effective overhanging flange width shall not exceed:

(a) one-twelfth the span length of the beam;

(b) six times the slab thickness;

(c) one-half the clear distance to the next web.

8.10.4 — Isolated beams, in which the T-shape is used to provide a flange for additional compression area, shall have a flange thickness not less than one-half the width of web and an effective flange width not more than four times the width of web.

8.10.5 — Where primary flexural reinforcement in a slab that is considered as a T-beam flange (excluding joist construction) is parallel to the beam, reinforcement perpendicular to the beam shall be provided in the top of the slab in accordance with the following:

8.10.5.1 — Transverse reinforcement shall be designed to carry the factored load on the overhanging slab width assumed to act as a cantilever. For isolated beams, the full width of overhanging flange shall be considered. For other T-beams, only the effective overhanging slab width need be considered.

8.10.5.2 — Transverse reinforcement shall be spaced not farther apart than five times the slab thickness, nor farther apart than 18 in.

8.11 — Joist construction

8.11.1 — Joist construction consists of a monolithic combination of regularly spaced ribs and a top slab arranged to span in one direction or two orthogonal directions.

8.11.2 — Ribs shall be not less than 4 in. in width, and shall have a depth of not more than 3-1/2 times the minimum width of rib.

COMMENTARY

R8.10 — T-beam construction

This section contains provisions identical to those of previous codes for limiting dimensions related to stiffness and flexural calculations. Special provisions related to T-beams and other flanged members are stated in 11.6.1 with regard to torsion.

R8.11 — Joist construction

The size and spacing limitations for concrete joist construction meeting the limitations of 8.11.1 through 8.11.3 are based on successful performance in the past.

DESIGN OF CONCRETE STRUCTURES

Twelfth Edition

Arthur H. Nilson

Professor Emeritus
Structural Engineering
Cornell University

With contributions by

David Darwin

Professor of Civil Engineering
University of Kansas

WCB McGraw-Hill

Boston, Massachusetts Burr Ridge, Illinois Dubuque, Iowa
Madison, Wisconsin New York, New York San Francisco, California St. Louis, Missouri

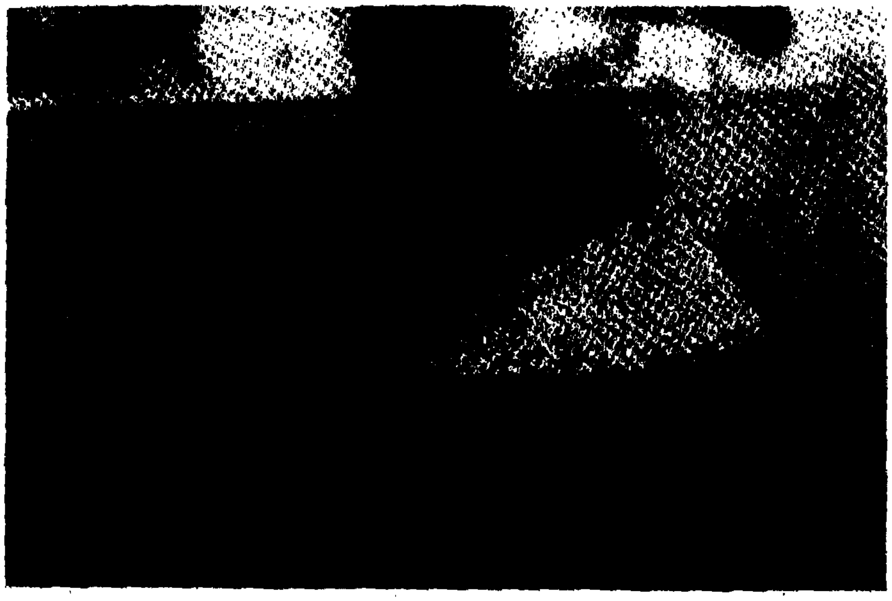

FIGURE 14.15
Development of corner levers in a simply supported, uniformly loaded slab.

considerably more complicated if the possibility of corner levers is introduced, and the error made by neglecting them is usually small.

To illustrate, the uniformly loaded square slab of Example 14.2, when analyzed for the assumed yield pattern of Fig. 14.7, required an ultimate moment capacity of $wL^2/24$. The actual yield line pattern at failure is probably as shown in Fig. 14.14*b*. Since two additional parameters m and n have necessarily been introduced to define the yield line pattern, a total of three equations of equilibrium is now necessary. These equations are obtained by summing moments and vertical forces on the segments of the slab. Such an analysis results in a required resisting moment of $wL^2/22$, an increase of about 9 percent compared with the results of an analysis neglecting corner levers. The influence of such corner effects may be considerably larger when the corner angle is less than 90°.

14.8 FAN PATTERNS AT CONCENTRATED LOADS

If a concentrated load acts on a reinforced concrete slab at an interior location, away from any edge or corner, a negative yield line will form in a more-or-less circular pattern, as in Fig. 14.16*a*, with positive yield lines radiating outward from the load point. If the positive resisting moment per unit length is m and the negative resisting moment m', the moments per unit length acting along the edges of a single element of the fan, having a central angle β and radius r, are as shown in Fig. 14.16*b*. For small values of the angle β, the arc along the negative yield line can be represented as a straight line of length $r\beta$.

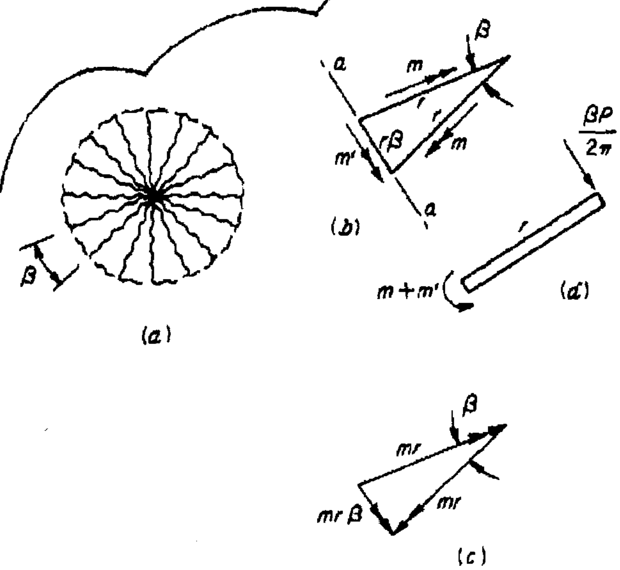

FIGURE 14.16
Yield fan geometry at concentrated load: (*a*) yield fan; (*b*) moment vectors acting on fan segment; (*c*) resultant of positive moment vectors; (*d*) edge view of fan segment.

Figure 14.16*c* shows the moment resultant obtained by vector addition of the positive moments mr acting along the radial edges of the fan segment. The vector sum is equal to $mr\beta$, acting along the length $r\beta$, and the resultant positive moment, per unit length, is therefore m. This acts in the same direction as the negative moment m', as shown in Fig. 14.16*d*. Figure 14.16*d* also shows the fractional part of the total load P that acts on the fan segment.

Taking moments about the axis $a - a$ gives

$$(m + m')r\beta - \frac{\beta P r}{2\pi} = 0$$

from which

$$\boxed{P = 2\pi(m + m')} \tag{14.4}$$

The collapse load P is seen to be independent of the fan radius r. With only a concentrated load acting, a complete fan of any radius could form with no change in collapse load.

It follows that Eq. (14.4) also gives the collapse load for a fixed-edge slab of any shape, carrying only a concentrated load P. The only necessary condition is that the boundary must be capable of a restraining moment equal to m' at all points.

Other load cases of practical interest, including a concentrated load near or at a free edge, and a concentrated corner load, are treated in Ref. 14.5. Loads distributed over small areas and load combinations are discussed in Ref. 14.12.

REINFORCED CONCRETE FUNDAMENTALS

FIFTH EDITION

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THE LATE T. U. TAYLOR PROFESSOR EMERITUS OF CIVIL ENGINEERING
THE UNIVERSITY OF TEXAS AT AUSTIN

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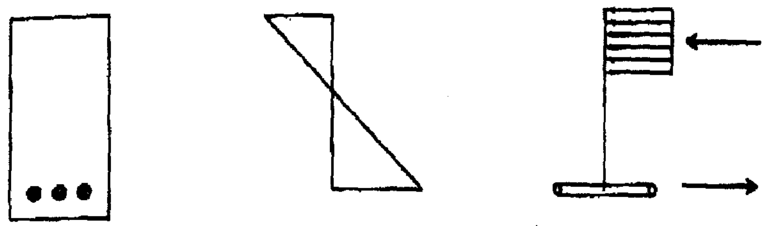
17

DISTRIBUTION OF CONCENTRATED LOADS AND OTHER SPECIAL PROBLEMS



17.1 Concrete Structures Distribute Concentrated Loads

The ordinary reinforced concrete structure is either monolithic or is tied together to act as a unit. Although parallel members of the structure may be analyzed somewhat independently of each other under uniform live loads, the entire structure is actually a three-dimensional frame. When moving concentrated loads are considered, their spacing and their number suggest that all parallel slab strips and all neighboring beams are not equally loaded. The interaction of the several slab strips and beams is usually such as to make the effective slab loading less severe than if each set of loads acted separately on the individual members.

When a heavy wheel rolls over a plank floor, each plank in turn must support the total load. In contrast, when a wheel moves over a concrete slab the wheel deflects the slab locally into a saucerlike pattern and this depression moves with the wheel across or along the slab. Thus a slab strip is deflected (and must be loaded) without a wheel actually resting on it. As the wheel passes over a particular strip the deflection increases, but the single 1-ft strip of slab never carries the entire wheel load unassisted. The designer describes this by saying the wheel load in Fig. 17.1*a* is distributed over an effective width E (Fig. 17.1*b*), that is, the moment on the most heavily loaded 1-ft strip is produced by $1/E$ parts of the total load, as in Fig. 17.1*c*. Likewise, closely spaced beams share in carrying concentrated loads when the beams are connected by stiff floor slabs or stiff diaphragms.

The result of a theoretical study of how a single wheel load is carried by a simple girder highway span is shown in Fig. 17.2. The load is applied to midspan, directly over beam B, and the assumed girder stiffness is five times that of the slab for a width equal to the girder span. Girder B then deflects more than its neighbors A and C. The slab (attached to the beams) is pulled down by beam B, but it resists this movement and exerts upward forces on the beam, as shown attempt to demonstrate how load distribution factors are established nor to ulate them for the many possible conditions. Rather its objective is to call ntion to the problem of load distribution and illustrate how it can be handled a few typical cases.

.2 Load Distribution in a Concrete Slab

oad distribution in a slab is approached on two different bases: (1) the service oad or deflection basis and (2) the ultimate strength or yield line basis. The istribution at service loads is the one more commonly considered. For a wide lab with a 10-ft simple span, the effective width thus determined for a simple oad is between 6 and 8 ft, depending somewhat on the size of the load contact rea and the particular algebraic formula[2] used. For comparison, Johansen has hown[3] that at ultimate load, the effective width is twice the span multiplied by $\sqrt{\mu}$, where μ is the ratio of the perpendicular top steel to the longitudinal bottom steel. If μ is about 1/3, the effective width is $2 \times 10\sqrt{0.33} = 11.5$ ft.

One complication that may make calculations at ultimate strength uncertain is the shear capacity of the slab around the load. Richart and Kluge[4] found shear failures occurring from diagonal tension, with a truncated cone of concrete punched out below the load. When those shear stresses were calculated on a surface at a distance d beyond the load, the unit shear stress was low, in one series from $0.044f'_c$ to $0.057f'_c$. Because the shear failures came at loads 50% greater than those producing local yielding of the steel, the low shear stresses were not considered serious. For a yield line analysis shear stresses around the load might be more significant.

It appears that the distribution based on elastic conditions, as commonly used, is on the safe side. Its use also tends to reduce crack size at working loads. For elastic conditions, Westergaard[5] established an extreme value of maximum positive moment on a slab as $0.315P$ for any simple span when P is distributed over a circular area with the diameter equal to $\frac{1}{10}$ of the span, the slab thickness is $\frac{1}{5}$ of the span, and Poisson's ratio is 0.15. (This local moment is quite sensitive to the size of the bearing area.) The corresponding transverse moment is $0.248P$. Jensen[6] extended these results to show the effect of a rigid beam support at right angles, that is, an effect similar to that in a two-way slab. At this crossbeam the maximum negative moment is $-P/2\pi = -0.159P$ and it occurs with the wheel quite close to the beam.

When closely spaced multiple wheels occurs, an extra slab width acts, but the effective width per wheel is reduced. The AASHTO *Standard Specifications for Highway Bridges*[7] specify such an effective width E for a slab carrying a single wheel (traveling in the direction of the span) that the resultant design is safe for multiple wheels without further calculations. Special transverse distribution steel is also specified as a percentage of the positive moment steel, in the amount of $100/\sqrt{S}$ but not over 50%, where S is the span in feet.* When the

* The AASHTO notations S and E are retained in this chapter.

Eclipse Engineering, Inc.
consulting engineers

March 11, 2004

Mr. Brady Orchard
Portage Environmental
591 Park Avenue, Suite 201
Idaho Falls, ID 83402

Re: Concrete Floor Analysis
Building TAN 607A, High Bay Assembly Shop
Idaho National Engineering & Environmental Laboratory
Idaho Falls, Idaho

Brady,

As requested, I have analyzed the concrete floor of the above noted building and I have determined its superimposed live load capacity. We have not analyzed the floor of the entire structure, but only the portion described in this letter.

The floor and foundation area bound by grid lines 1 to the west, 8 to the east, N to the north and P to the south is a system of cast-in-place concrete slabs, grade beams and drilled concrete piers. This area, also known as the HIGH BAY ASSEMBLY SHOP, is approximately 58'-0" x 147'-0" as shown on the attached PARTIAL FOUNDATION & FIRST FLOOR PLAN. Also reference the original structural drawings produced by The Ralph M. Parsons Company, dated 8/3/56.

Within the area described above, there are several different sub-areas where the configuration of the slab, grade beams and drilled piers vary as follows (also reference the attached KEY PLAN):

Sub-area 1: Defined roughly as between grids N and N.2, 1 and 8. This sub-area is an 8-inch thick slab-on-grade.

Sub-area 2: Defined roughly as between grids N.2 and N.6, 1 and 4. This sub-area is a system of slabs, grade beams and drilled piers. Contained within this sub-area is the ASSEMBLY PIT

Sub-area 3: Defined roughly as between grids N.6 and P, 1 and 5. This sub-area is an 8-inch thick slab-on-grade.

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Sub-area 4: Defined roughly as between grids N.2 and N.6, 4 and 7.8. This sub-area is a system of slabs, grade beams and drilled piers

Sub-area 5: Defined roughly as between grids N.6 and P, 5 and 7. This sub-area is an 8-inch thick slab-on-grade supplemented with drilled piers. Contained within this sub-area is the BED PLATE.

Sub-area 6: Defined roughly as between grids N.2 and N.6, 7.8 and 8. This sub-area is an 8-inch thick slab-on-grade.

Sub-area 7: Defined roughly as between grids N.6 and P, 7 and 8. This sub-area is an 8-inch thick slab-on-grade supplemented with grade beams and drilled piers.

We have determined the superimposed live load capacity of each sub-area as follows (reference our calculation booklet for the detailed structural analysis):

Sub-area 1: 500 pounds per square foot.
Sub-area 2: 2615 pounds per square foot, except the assembly pit which is limited to 285 pounds per square foot.
Sub-area 3: 500 pounds per square foot.
Sub-area 4: 1542 pounds per square foot.
Sub-area 5: 1490 pounds per square foot.
Sub-area 6: 500 pounds per square foot.
Sub-area 7: 500 pounds per square foot, except the 6'-2" wide strip that supports the railroad tracks which can support up to 1895 pounds per square foot.

We have analyzed only the concrete floor of the HIGH BAY ASSEMBLY SHOP as described in this letter. We hold no responsibility for any other element or the integrity of the structure as a whole. Please call with any specific questions.

Sincerely,

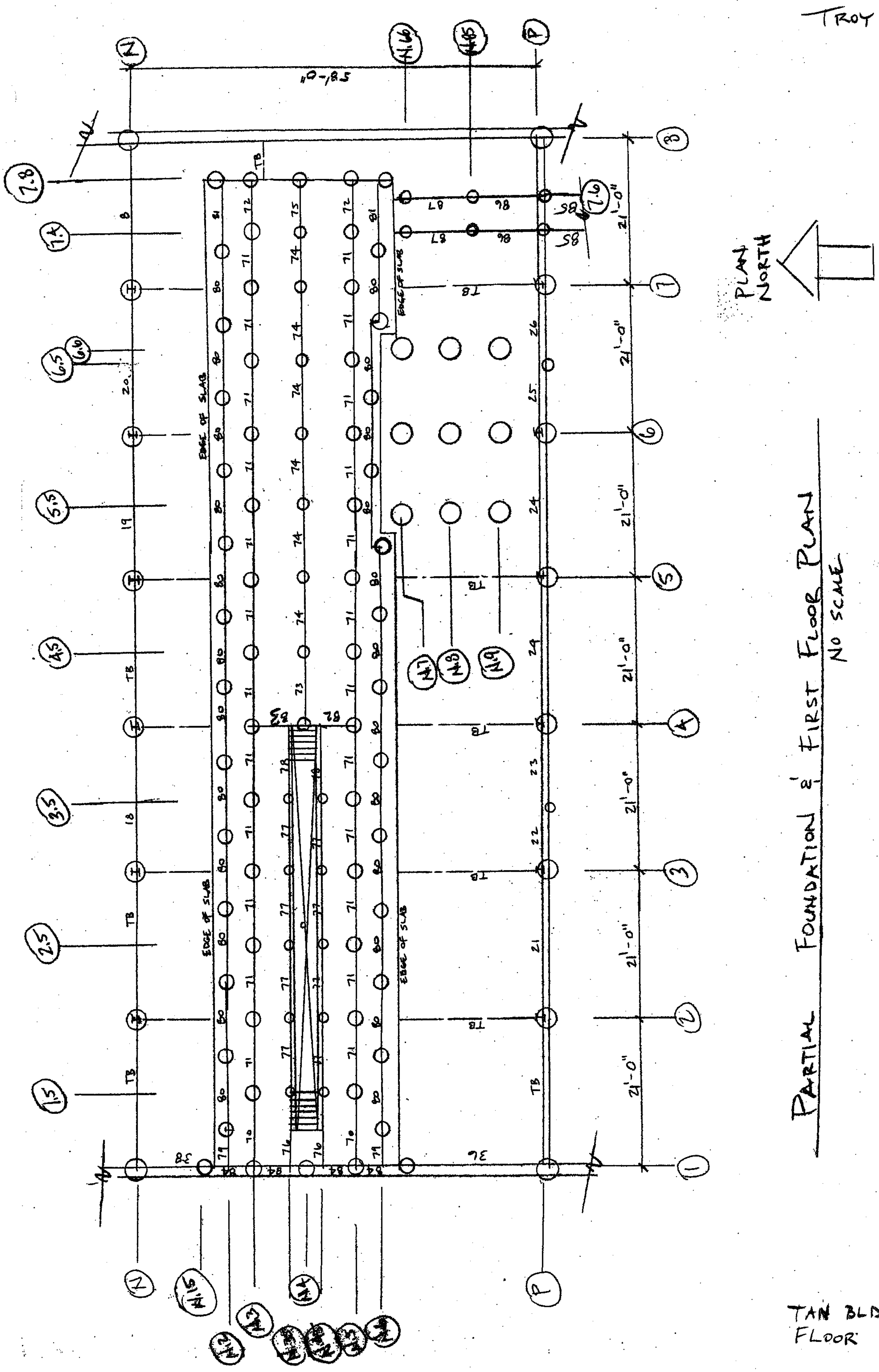
Eclipse Engineering, Inc.

PROFESSIONAL ENGINEER
REGISTERED
9081
3/15/04
STATE OF IDAHO
TROY LEISTIKO

Troy Leistiko, P.E.
Project Engineer

Attachments: PARTIAL FOUNDATION & FIRST FLOOR PLAN, KEY PLAN

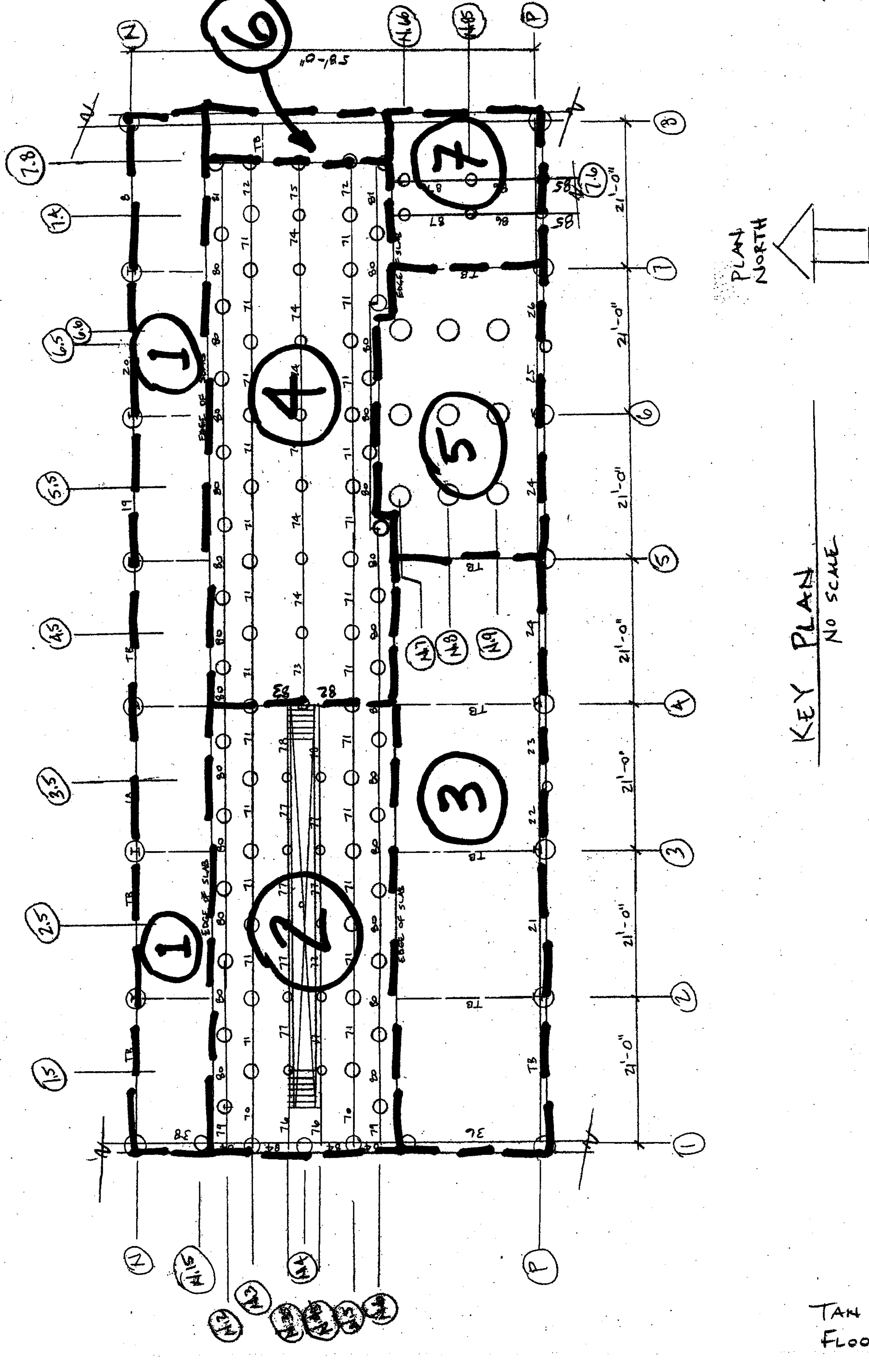
Eclipse Engineering
Troy Leistiko
3/11/04



PARTIAL FOUNDATION & FIRST FLOOR PLAN
NO SCALE

TAN BLDG. 607A
FLOOR ANALYSIS

Eclipse Engineerin
Troy Leistiko
3/11/04



Tan Bldg. 607A
Floor Analysis

Eclipse Engineering, Inc.
consulting engineers

Structural Calculations

Concrete Floor Analysis
Building TAN 607A, High Bay Assembly Shop
Idaho National Engineering & Environmental Laboratory
Idaho Falls, Idaho

PROFESSIONAL ENGINEER
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3/15/04
STATE OF IDAHO
TROY LEISTIKO

Prepared For:
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Idaho Falls, ID 83402

structural mechanics

235 N. 1st St. West, 2nd Floor
Missoula, MT 59802
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Structural Narrative:

The floor and foundation area bound by grid lines 1 to the west, 8 to the east, N to the north and P to the south is a system of cast-in-place concrete slabs, grade beams and drilled concrete piers. This area, also known as the HIGH BAY ASSEMBLY SHOP, is approximately 58'-0" x 147'-0" as shown on the attached PARTIAL FOUNDATION & FIRST FLOOR PLAN. Also reference the original structural drawings produced by The Ralph M. Parsons Company, dated 8/3/56.

Within the area described above, there are several different sub-areas where the configuration of the slab, grade beams and drilled piers vary as follows (also reference the attached KEY PLAN):

Sub-area 1: Defined roughly as between grids N and N.2, 1 and 8. This sub-area is an 8-inch thick slab on grade.

Sub-area 2: Defined roughly as between grids N.2 and N.6, 1 and 4. This sub-area is a system of slabs, grade beams and drilled piers. Contained within this sub-area is the ASSEMBLY PIT

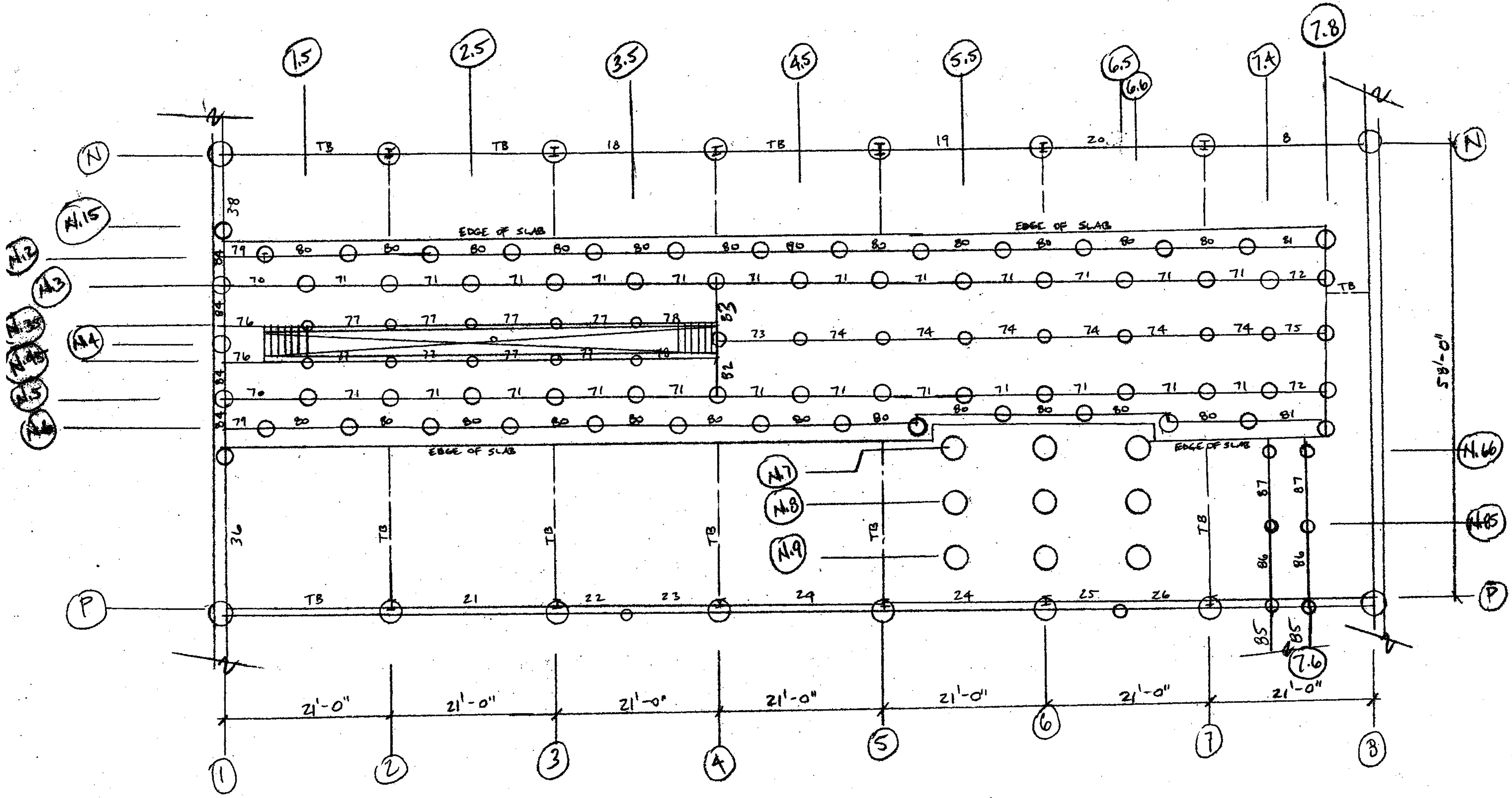
Sub-area 3: Defined roughly as between grids N.6 and P, 1 and 5. This sub-area is an 8-inch thick slab on grade.

Sub-area 4: Defined roughly as between grids N.2 and N.6, 4 and 7.8. This sub-area is a system of slabs, grade beams and drilled piers.

Sub-area 5: Defined roughly as between grids N.6 and P, 5 and 7. This sub-area is an 8-inch thick slab-on-grade supplimented with drilled piers. Contained within this sub-area is the BED PLATE.

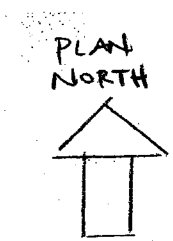
Sub-area 6: Defined roughly as between grids N.2 and N.6, 7.8 and 8. This sub-area is an 8-inch thick slab on grade.

Sub-area 7: Defined roughly as between grids N.6 and P, 7 and 8. This sub-area is an 8-inch thick slab-on-grade supplimented with grade beams and drilled piers.



PARTIAL FOUNDATION & FIRST FLOOR PLAN

NO SCALE



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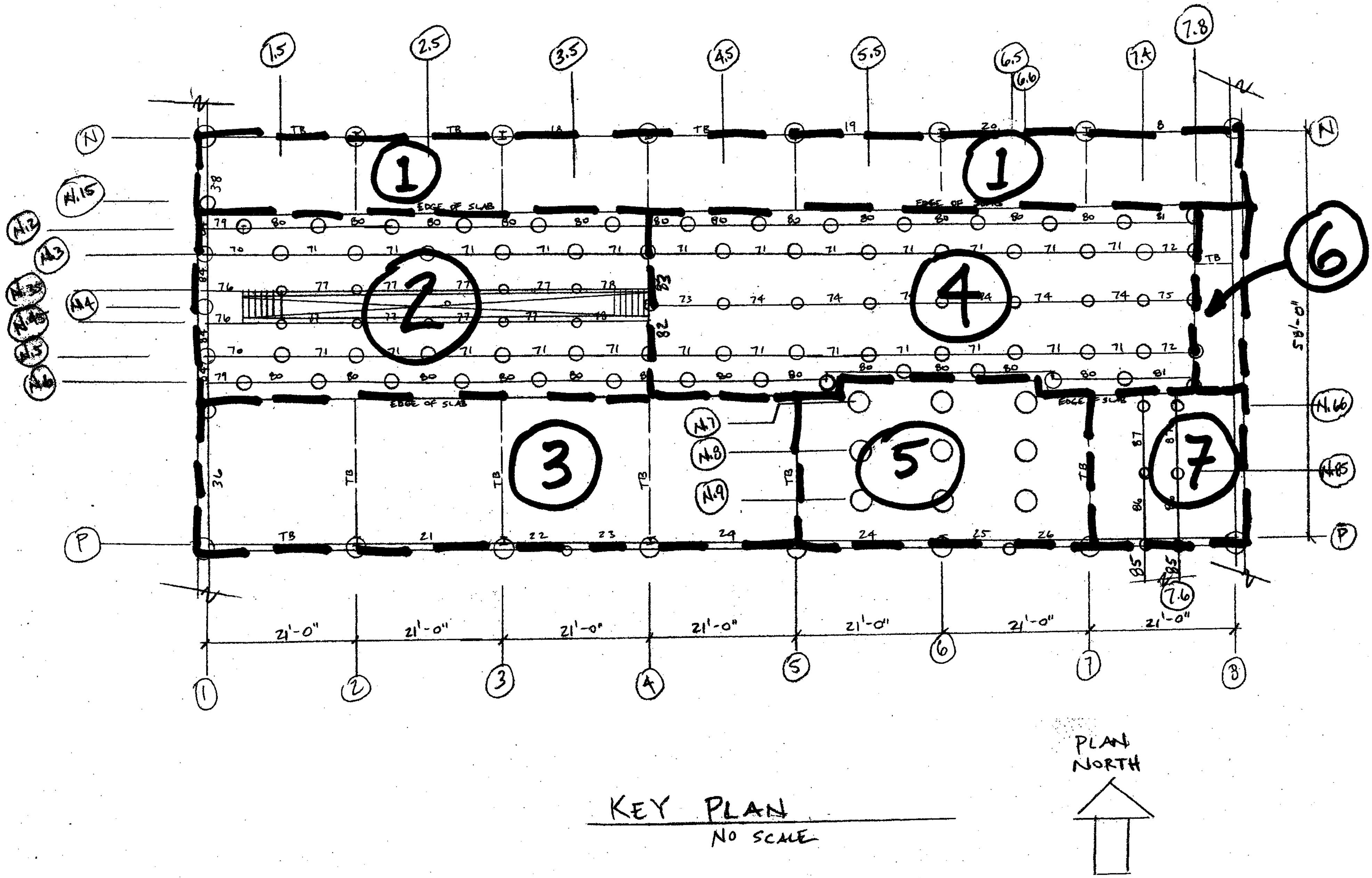

KEY PLAN
NO SCALE

3/27

Grade Beam Table

Label	Ctr. To Ctr. Span, (ft.)	width, b (in.)	height, h (in.)	depth to rebar, d (in.)	BOTTOM BARS qty.	size		A_s (in^2)	TOP BARS qty.	size		A_s' (in^2)	STIRRUPS size		A_v (in^2)	spacing (in.)	STRENGTH $\phi_b M_n^+$ (k-ft)	$\phi_b M_n^-$ (k-ft)	$\phi_v V_n$ (k)
70	10.50	26	66	62	6	# 9	=	6.00	8	# 7	=	4.80	# 4	=	0.40	12	1077	868	207
71	10.50	26	66	62	5	# 8	=	3.95	8	# 7	=	4.80	# 4	=	0.40	12	718	868	207
72	7.75	26	66	62	6	# 9	=	6.00	8	# 7	=	4.80	# 4	=	0.40	12	1077	868	207
73	10.50	18	48	44	5	# 8	=	3.95	6	# 7	=	3.60	# 4	=	0.40	12	497	455	117
74	10.50	18	48	44	5	# 8	=	3.95	6	# 7	=	3.60	# 4	=	0.40	12	497	455	117
75	7.75	18	48	44	5	# 8	=	3.95	6	# 7	=	3.60	# 4	=	0.40	12	497	455	117
76	10.50	12	66	62	3	# 8	=	2.37	3	# 9	=	3.00	# 4	=	0.40	12	428	537	134
77	10.50	12	66	62	3	# 8	=	2.37	3	# 9	=	3.00	# 4	=	0.40	12	428	537	134
78	10.50	12	66	62	3	# 8	=	2.37	3	# 9	=	3.00	# 4	=	0.40	12	428	537	134
79	5.25	66	36	32	10	# 10	=	12.70	20	# 6	=	8.80	# 4	=	0.40	12	1150	812	216
80	10.50	66	36	32	8	# 10	=	10.16	20	# 6	=	8.80	# 4	=	0.40	12	931	812	216
81	10.50	66	36	32	10	# 10	=	12.70	20	# 6	=	8.80	# 4	=	0.40	12	1150	812	216
82	7.50	15	66	62	4	# 8	=	3.16	4	# 8	=	3.16	# 4	=	0.40	12	569	569	149
83	7.50	15	66	62	4	# 8	=	3.16	4	# 8	=	3.16	# 4	=	0.40	12	569	569	149
84	8.00	18	66	62	2	# 9	=	2.00	2	# 9	=	2.00	# 4	=	0.40	12	366	366	165
85	10.00	20	35	31	4	# 7	=	2.40	3	# 8	=	2.37	# 4	=	0.40	12	215	212	88
86	9.50	20	35	31	4	# 7	=	2.40	3	# 8	=	2.37	# 4	=	0.40	12	215	212	88
87	9.50	20	35	31	4	# 7	=	2.40	3	# 8	=	2.37	# 4	=	0.40	12	215	212	88
TB	N/A	16	24	20	2	# 5	=	0.62	2	# 5	=	0.62	N/ A	=	0.00	12	37	37	27

Rebar Area Sub-Table

Bar Size	Area (in^2)
3	0.11
4	0.20
5	0.31
6	0.44
7	0.60
8	0.79
9	1.00
10	1.27
11	1.56

Properties & Formulae

f'_c = 2500 psi

F_y = 40000 psi

F_v = 40000 psi

ϕ_b = 0.9

ϕ_v = 0.85

$a = \frac{A_s * F_y}{0.85 * f'_c * b}$ OR $\frac{A_s' * F_y}{0.85 * f'_c * b}$

$M_n^+ = A_s * F_y * (d - a/2)$

$M_n^- = A_s' * F_y * (d - a/2)$

$V_n = (V_c + A_v * F_v * d/s)$

where $V_c = 2 * \mathrm{sqrt}(f'_c) * b * d$

4/27

SAMPLE CALCULATION (UNABRIDGED)

Label : Grade Beam 70, M+

CONCRETE BEAMS

$k := 1000 \cdot lb$ $\quad$ $plf := \frac{lb}{ft}$ $\quad$ $psi := \frac{lb}{in^2}$

$\phi_b := 0.9$ $\quad$ $\phi_v := 0.85$

$b := 26 \cdot in$ $\quad$ $h := 66 \cdot in$ $\quad$ $d := h - 4 \cdot in$ $\quad$ $d = 62\,in$

$f_c := 2500 \cdot psi$ $\quad$ $f_y := 40000 \cdot psi$ $\quad$ $f_v := 40000 \cdot psi$

$E := 57000 \cdot \sqrt{f_c} \cdot \frac{lb^{\frac{1}{2}}}{in}$ $\quad$ $E = 2850000 \frac{lb}{in^2}$ $\quad$ $I_g := \frac{b \cdot h^3}{12}$

$\rho_{min} := \frac{200 \cdot psi}{f_y}$ $\quad$ $A_{min} := \rho_{min} \cdot b \cdot d$ $\quad$ $A_{min} = 8.06\,in^2$

Analyze (6) #9 Bars bottom reinf.
f'c = 2,500 psi, fy = 40,000 psi
#4 stirrups @ 12" o.c.

$A := 6.00 \cdot in^2$ $\quad$ $A = 6\,in^2$

$\rho := \frac{A}{b \cdot d}$ $\quad$ $\rho = 0.00372$ $\quad$ $\rho_{min} = 0.005$

$a := \frac{A \cdot f_y}{0.85 \cdot f_c \cdot b}$ $\quad$ $M_d := \phi_b \cdot A \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $\quad$ $\phi M_d = 1076.905\,k \cdot ft$

$A_v := 0.40 \cdot in^2$ $\quad$ $V_c := 2 \cdot \sqrt{f_c} \cdot b \cdot d \cdot \frac{lb^{\frac{1}{2}}}{in}$ $\quad$ $V_c = 161.2\,k$

$s := 12 \cdot in$ $\quad$ $\phi V_d := \phi_v \cdot \left(V_c + \frac{A_v \cdot f_v \cdot d}{s}\right)$ $\quad$ $\phi V_d = 207.3\,k$

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ANALYSIS NOTES

EFF. SPAN =
- CENTER TO CENTER OF SUPPORTS FOR ANALYSIS
- DESIGN AT FACES OF SUPPORTS (MOMENTS)
- CLEAR SPAN ANALYSIS FOR THE SLABS.
- DESIGN FOR V_u AT d FROM FACE OF SUPPORT

CODE

COMMENTARY

to the structure should be considered in the analysis of the structure because they may lead to increased design forces in some or all elements. Special provisions for seismic design are given in Chapter 21.

8.2.4 — Consideration shall be given to effects of forces due to prestressing, crane loads, vibration, impact, shrinkage, temperature changes, creep, expansion of shrinkage-compensating concrete, and unequal settlement of supports.

R8.2.4 — Information is accumulating on the magnitudes of these various effects, especially the effects of column creep and shrinkage in tall structures,[8.1] and on procedures for including the forces resulting from these effects in design.

8.3 — Methods of analysis

R8.3 — Methods of analysis

8.3.1 — All members of frames or continuous construction shall be designed for the maximum effects of factored loads as determined by the theory of elastic analysis, except as modified according to 8.4. It shall be permitted to simplify design by using the assumptions specified in 8.6 through 8.9.

R8.3.1 — Factored loads are service loads multiplied by appropriate load factors. If the alternate design method of Appendix A is used, the loads used in design are service loads (load factors of unity). For both the strength design method and the alternate design method, elastic analysis is used to obtain moments, shears, and reactions.

8.3.2 — Except for prestressed concrete, approximate methods of frame analysis shall be permitted for buildings of usual types of construction, spans, and story heights.

8.3.3 — As an alternate to frame analysis, the following approximate moments and shears shall be permitted for design of continuous beams and one-way slabs (slabs reinforced to resist flexural stresses in only one direction), provided:

(a) There are two or more spans;

(b) Spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20 percent;

(c) Loads are uniformly distributed;

(d) Unit live load does not exceed three times unit dead load; and

(e) Members are prismatic.

Positive moment	
End spans	
Discontinuous end unrestrained	$w_u \ell_n^2/11$
Discontinuous end integral with support	$w_u \ell_n^2/14$
Interior spans	$w_u \ell_n^2/16$

R8.3.3 — The approximate moments and shears give reasonably conservative values for the stated conditions if the flexural members are part of a frame or continuous construction. Because the load patterns that produce critical values for moments in columns of frames differ from those for maximum negative moments in beams, column moments should be evaluated separately.

ℓ_n = clear span

7/27

CODE

Negative moments at exterior face
of first interior support

Two spans .... $w_u \ell_n^2/9$
More than two spans .... $w_u \ell_n^2/10$

Negative moment at other faces
of interior supports .... $w_u \ell_n^2/11$

Negative moment at face of all
supports for

Slabs with spans not exceeding
10 ft; and beams where ratio of
sum of column stiffnesses to
beam stiffness exceeds eight at
each end of the span .... $w_u \ell_n^2/12$

Negative moment at interior face
of exterior support for members
built integrally with supports

Where support is spandrel beam .... $w_u \ell_n^2/24$
Where support is a column .... $w_u \ell_n^2/16$

Shear in end members at face of
first interior support .... $1.15\ w_u \ell_n/2$

Shear at face of all other
supports .... $w_u \ell_n/2$

COMMENTARY

ℓ_n = clear span

8.4 — Redistribution of negative moments in continuous nonprestressed flexural members

For criteria on moment redistribution for prestressed concrete members, see 18.10.4.

8.4.1 — Except where approximate values for moments are used, it shall be permitted to increase or decrease negative moments calculated by elastic theory at supports of continuous flexural members for any assumed loading arrangement by not more than

$$20\left(1 - \frac{\rho - \rho'}{\rho_b}\right) \text{ percent}$$

8.4.2 — The modified negative moments shall be used for calculating moments at sections within the spans.

R8.4 — Redistribution of negative moments in continuous nonprestressed flexural members

Moment redistribution is dependent on adequate ductility in plastic hinge regions. These plastic hinge regions develop at points of maximum moment and cause a shift in the elastic moment diagram. The usual result is a reduction in the values of negative moments in the plastic hinge region and an increase in the values of positive moments from those computed by elastic analysis. Because negative moments are determined from one loading arrangement and positive moments from another, each section has a reserve capacity that is not fully utilized for any one loading condition. The plastic hinges permit the utilization of the full capacity of more cross sections of a flexural member at ultimate loads.

Using conservative values of ultimate concrete strains and lengths of plastic hinges derived from extensive tests, flexural members with small rotation capacity were analyzed for moment redistribution varying from 10 to 20 percent, depending on the reinforcement ratio. The results were found to be conservative (see Fig. R8.4). Studies by Cohn[8.2] and Mattock[8.3] support this conclusion and indicate that

8/21

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SUB-AREAS 1, 3, 6

EACH OF THESE SUB-AREAS IS AN 8" THICK SLAB-ON-GRADE THAT IS REINFORCED W/ 4"×4" - #6/#6 WELDED WIRE FABRIC.

IN ACCORDANCE W/ ACI 807, THE CAPACITY FOR THIS TYPE SLAB IS 500 PSF SUPERIMPOSED LIVE LOAD. (SEE ATTACHED TABLE)

CAPACITY = 500 PSF LIVE LOAD

SLABS ON GROUND *

For any slab on the ground, adequate preparation of subgrade for drainage and compaction is of prime importance. Dowelled expansion joints and weakened plane contraction joints should be carefully located, including expansion joints at all walls.

The design of slabs on the ground to distribute concentrated or uniform loads involves the elastic properties of the subsoil and the slab itself. An analysis can be made but is quite involved. Slabs for the very lightest occupancy should be not less than 4″ thick, and slabs for other occupancies may be empirically selected, the following being about minimum and sometimes less than what is required by ACI 807 for supported slabs:—

Occupancy **	Min. Slab Thickness	Reinforcement ‡
Sub-slabs under other slabs	2″	None
Domestic or light commercial (loaded less than 100 psf)	4″	One layer 6 x 6 10/10 welded wire fabric, minimum for ideal conditions: 6 x 6 8/8 for average conditions.
Commercial—institutional—barns (loaded 100-200 psf)	5″	One layer 6 x 6 8/8 welded wire fabric or one layer 6 x 6 6/6.
Industrial (loaded not over 400-500 psf) and pavements for industrial plants, gas stations, and garages	6″	One layer 6 x 6 6/6 welded wire fabric or one layer 6 x 6 4/4.
Industrial (loaded 600–800 psf) and heavy pavements for industrial plants, gas stations, and garages	6″	Two layers 6 x 6 6/6 welded wire fabric or two layers 6 x 6 4/4
Industrial (loaded 1500 psf) †	7″	Two mats of bars (one top, one bottom), each of #4 bars @ 12″ c/c, each way
Industrial (loaded 2500 psf) †	8″	Two mats of bars (one top, one bottom), each of #5 bars @ 12″ c/c, each way
Industrial (loaded 3000–3500 psf) †	9″	Two mats of bars (one top, one bottom), each of #5 bars @ 8″ to 12″ c/c, each way

* For further details, see "Concrete Floors on Ground," and "Concrete Airport Pavement," Portland Cement Association, 33 West Grand Avenue, Chicago, Illinois, 1952, and "Design of Concrete Floors on Ground for Warehouse Loadings," Aug. 1957 Journal, American Concrete Institute, P. O. Box 4754, Redford Sta., Detroit 19, Mich.

** For loads in excess of, say, 500 psf, use at least 3000 psi quality controlled concrete, and investigate subsoil conditions with extra care. Fill material and compaction should be equivalent to ordinary highway practice. If laboratory control of compaction is available, the load capacities can be increased in the ratio of the actual compaction coefficient, k, to 100.

† For loads in excess of, say, 1500 psf the subsoil conditions should be investigated with extra care.

‡ Place first layer of reinforcement 2 in. below top of slab; second layer, 2 in. up from bottom of slab.

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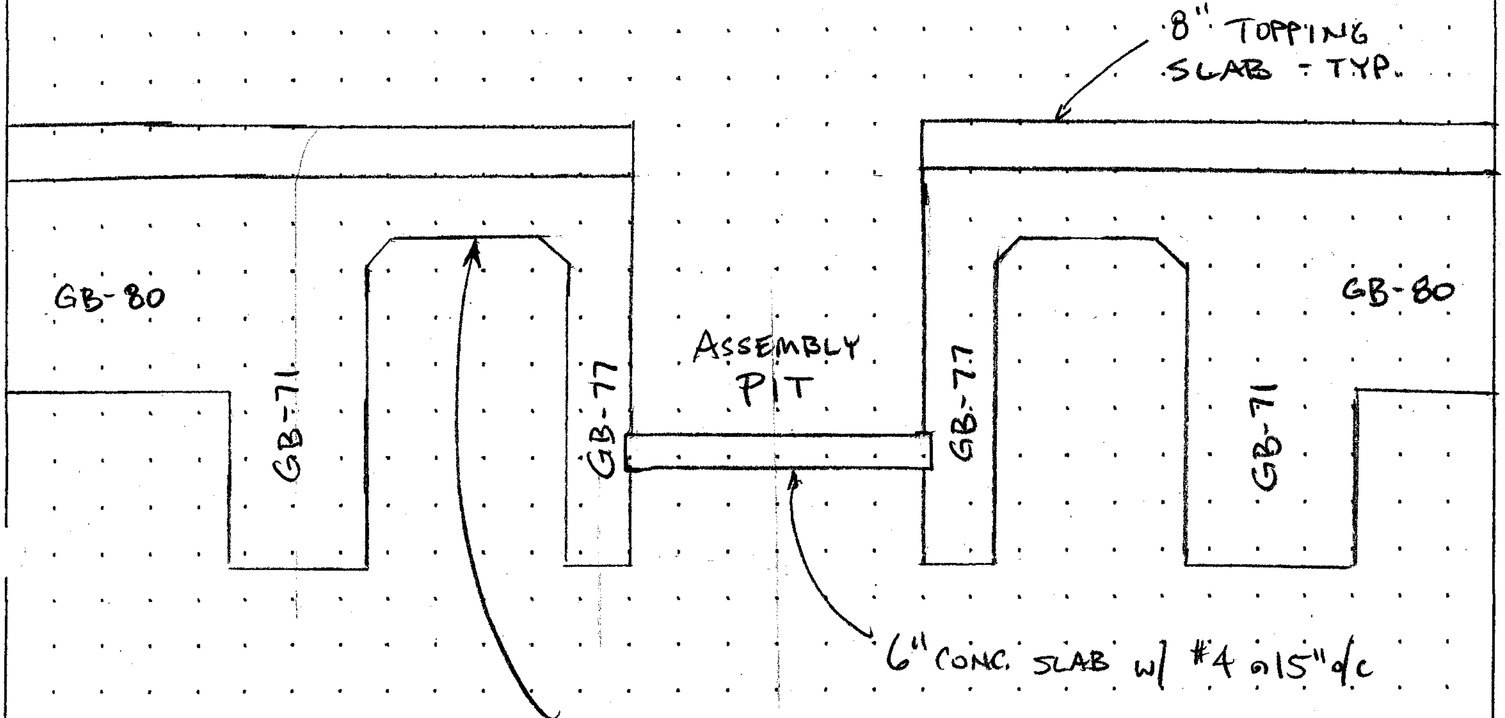
TAN BLDG 607A
FLOOR ANALYSIS
TROY 3/4/04

SUB-AREA 2 - REF. A / S115



10" CONC SLAB w/ #6 @ 10" o.c.
THAT IS 15" DEEP AT THE GRADE BEAMS
d to $A_s = 7"$
d to $A'_s = 13"$

TYPICAL CROSS SECTION
NO SCALE

*CAPACITY = 2615 PSF LIVE LOAD

* EXCEPT THE ASSEMBLY PIT WHICH IS LIMITED TO 285 PSF LIVE LOAD

C.I.P. Concrete Beam or Slab Analysis - Determine max. superimposed (live) load allowed

Beam Label	Assembly Pit, 6" slab	Typical 15" slab	70	71	72
Depth of Beam (in) **h**	6	10	66	66	66
Depth to Reinf. (in) **d**	4	7	62	62	62
Width of Beam (in) **b**	15	10	26	26	26
Slab Section or Beam Size	**15 x 6**	**10 x 10**	**26 x 66**	**26 x 66**	**26 x 66**
Design Criteria					
	360	360	360	360	360
Δ limit due to Long-Term Loads (L /) applied after non-structural elements are attached	480	480	480	480	480
% of live load that is long-term	20%	20%	20%	20%	20%
% of live load that is not long-term	80%	80%	80%	80%	80%
$\lambda = \xi/(1+50p')$, ξ = 2.0 for long-term lo	2.00	1.52	1.74	2.00	2.00
Concrete unit weight (pcf)	150	150	150	150	150
Floor Uniform Dead Load (psf)	75	100	100	100	100
Floor Uniform Live Load (psf)	285	2615	5000	5000	5000
Floor Beam Linear Dead Load (plf)	93.75	104.17	1787.50	1787.50	1787.50
Analysis, ref. ACI 318-99, sections 8.7 (span length), 8.3 (methods of analysis), and 11.1.3.1					
Span (Ctr to Ctr of Supports) (ft)	6	5	10.5	10.5	7.75
Width of Supports (in)	12	24	26	26	26
Analyze Ctr-Ctr(0) or Clr Span(1)	1	1	1	1	1
Effective Span (ft)	5	3	8.33333333	8.33333333	5.58333333
Tributary width (ft)	1.25	0.83	4.50	4.50	4.50
Include beam wt? No(0)/Yes(1)	0	1	1	1	1
Uniform Dead Load (plf)	93.75	187.47	2237.50	2237.50	2237.50
Uniform Live Load (plf)	356.25	2178.30	22500.00	22500.00	22500.00
U = 1.4D + 1.7L (plf)	737	3966	41383	41383	41383
V.u (lb), $1.15\omega_u\ell_n/2$	**2,119**	**6,841**	**198,291**	**198,291**	**132,855**
V.u (lb), $\omega_u\ell_n/2$	**1,842**	**5,948**	**172,427**	**172,427**	**115,526**
V.u (lb), $\omega_u\ell_n/2 - d\omega_u$	**1,597**	**3,635**	**-41,383**	**-41,383**	**-98,283**
Choose V.u	1,842	5,948	198,291	172,427	115,526
M^+.u (lb.ft), $\omega_u\ell_n/8$	**2,303**	**4,461**	**359,223**	**359,223**	**161,255**
M^+.u (lb.ft), $\omega_u\ell_n/11$	**1,675**	**3,245**	**261,253**	**261,253**	**117,277**
M^+.u (lb.ft), $\omega_u\ell_n/14$	**1,316**	**2,549**	**205,270**	**205,270**	**92,146**
M^+.u (lb.ft), $\omega_u\ell_n/16$	**1,151**	**2,231**	**179,612**	**179,612**	**80,628**
Choose M^+.u	2,303	2,231	261,253	179,612	80,628
M^-.u (lb.ft), $\omega_u\ell_n/9$	**2,047**	**3,966**	**319,309**	**319,309**	**143,338**
M^-.u (lb.ft), $\omega_u\ell_n/10$	**1,842**	**3,569**	**287,378**	**287,378**	**129,004**
M^-.u (lb.ft), $\omega_u\ell_n/11$	**1,675**	**3,245**	**261,253**	**261,253**	**117,277**
M^-.u (lb.ft), $\omega_u\ell_n/12$	**1,535**	**2,974**	**239,482**	**239,482**	**107,503**
M^-.u (lb.ft), $\omega_u\ell_n/16$	**1,151**	**2,231**	**179,612**	**179,612**	**80,628**
M^-.u (lb.ft), $\omega_u\ell_n/24$	**768**	**1,487**	**119,741**	**119,741**	**53,752**
	0	**0**	**0**	**0**	**0**
Choose M^-.u	0	2,974	287,378	261,253	117,277

CONTINUED

Strength Design	**Pit, 6" slab**	**15" slab**	**70**
Flexural Steel Bars (Bottom)	(1) #4	(1) #6	(6) #9
Flexural Steel Area (in^2), **As**	0.20	0.44	6.00
Shear Steel Bars	None	None	(2) #4
Shear Steel Area (in^2), **Av**	0.00	0.00	0.40
spacing of shear steel (in), **s**	999	999	12
Flexural Steel Bars (Top)	None	(1) #6	(8) #7
Flexural Steel Area (in^2), **A's**	0.00	0.44	4.80
Concrete Strength (psi), f'c	2500	2500	2500
Flexural Steel (psi), fy	40000	40000	40000
Shear Steel (psi), fy	40000	40000	40000
Depth of top comp. block (in), **a**	0.25	0.83	4.34
ρ	0.00333	0.00629	0.00372
ρ,min	0.00500	0.00500	0.00500
Min. reinf. Check	more steel	OK	more steel
β_1	0.85	0.85	0.85
ρ,max = 0.75*ρ,balanced	0.02320	0.02320	0.02320
Max. reinf. Check	OK	OK	OK
Depth of bottom comp. block (in), **a**	0.00	0.83	3.48
ρ' (manual check min. & max.)	0.00000	0.00629	0.00298
ϕ.b	0.9	0.9	0.9
ϕ.v	0.85	0.85	0.85
Bending Strength, M^+.d = ϕM^+.n (lb*ft)	**2,325**	**8,693**	**1,076,905**
Check bending strength	OKAY	OKAY	OKAY
Bending Strength, M^-.d = ϕM^-.n (lb*ft)	**0**	**8,693**	**867,779**
Check bending strength	OKAY	OKAY	OKAY
Shear Strength, V.d = ϕV.n (lb)	**5,100**	**5,950**	**207,287**
Check shear strength	OKAY	OKAY	OKAY
Deflection Design (Valid for simple spans only)			
f.r, modulus of rupture (psi)	375	375	375
I.g, Gross moment of inertia (in^4)	270	833	622,908
y.t, distance from N.A. to tension face	3.00	5.00	33.00
M.cr, Cracking Moment (lb*ft)	2,813	5,208	589,875
M.max, Service Moment	1,406	2,661	214,735
E.s, Elastic Mod. Of Steel (psi)	29,000,000	29,000,000	29,000,000
E.c, Elastic Mod. Of concrete (psi)	2,850,000	2,850,000	2,850,000
n = E.s/E.c	10.2	10.2	10.2
c = d[(nρ*(nρ+2))^1/2-nρ]	0.91	2.10	14.88
I.cr, Cracked moment of inertia (in^4)	23	138	164,108
I.e, Effective moment of intertia (in^4)	**270**	**833**	**622,908**
Δ, immediate due to live load (in)	0.007	0.002	0.001
Span / deflection	**9216**	**21537**	**72716**
Check live deflection	OKAY	OKAY	OKAY
Δ, long term from dead load (in)	0.003	0.000	0.000
Δ, lg. term from sustained live ld. (in)	0.004	0.001	0.001
Δ, instantaneous live load (in)	0.005	0.001	0.001
Δ, after attachment of non-structural elements (in.) = Rows 95+96+97	0.013	0.002	0.002
Span / deflection	**4784**	**15806**	**52579**
Check live deflection	OKAY	OKAY	OKAY

C.I.P. Concrete Beam or Slab Analysis - Determine max. superimposed (live) load allowed

Beam Label	76	77	78	79	80
Depth of Beam (in) **h**	66	66	66	36	36
Depth to Reinf. (in) **d**	62	62	62	32	32
Width of Beam (in) **b**	12	12	12	66	66
Slab Section or Beam Size	12 x 66	12 x 66	12 x 66	66 x 36	66 x 36
Design Criteria					
	360	360	360	360	360
Δ limit due to Long-Term Loads (L /) applied after non-structural elements are attached	480	480	480	480	480
% of live load that is long-term	20%	20%	20%	20%	20%
% of live load that is not long-term	80%	80%	80%	80%	80%
$\lambda = \xi/(1+50\rho')$, $\xi = 2.0$ for long-term	1.66	1.66	1.66	2.00	2.00
Concrete unit weight (pcf)	150	150	150	150	150
Floor Uniform Dead Load (psf)	100	100	100	100	100
Floor Uniform Live Load (psf)	2933	3405	2933	5000	5000
Floor Beam Linear Dead Load (plf)	825.00	825.00	825.00	2475.00	2475.00
Analysis, ref. ACI 318-99, sections 8.7 (span length), 8.3 (methods of analysis), and 11.1.3.1					
Span (Ctr to Ctr of Supports) (ft)	10.5	10.5	10.5	5.25	10.5
Width of Supports (in)	22	22	22	28	28
Analyze Ctr-Ctr(0) or Clr Span(1)	1	1	1	1	1
Effective Span (ft)	8.66666667	8.66666667	8.66666667	2.91666667	8.16666667
Tributary width (ft)	5.00	5.00	5.00	4.00	4.00
Include beam wt? No(0)/Yes(1)	1	1	1	1	1
Uniform Dead Load (plf)	1325.00	1325.00	1325.00	2875.00	2875.00
Uniform Live Load (plf)	14665.00	17025.00	14665.00	20000.00	20000.00
U = 1.4D + 1.7L (plf)	26786	30798	26786	38025	38025
V.u (lb), $1.15\omega_u l_n/2$	**133,481**	**153,474**	**133,481**	**63,771**	**178,559**
V.u (lb), $\omega_u l_n/2$	**116,071**	**133,456**	**116,071**	**55,453**	**155,269**
V.u (lb), $\omega_u l_n/2 - d\omega_u$	**-22,321**	**-25,665**	**-22,321**	**-45,947**	**53,869**
Choose V.u	133,481	133,456	133,481	63,771	155,269
M^+.u (lb.ft), $\omega_u l_n/8$	**251,486**	**289,154**	**251,486**	**40,435**	**317,007**
M^+.u (lb.ft), $\omega_u l_n/11$	**182,899**	**210,294**	**182,899**	**29,407**	**230,551**
M^+.u (lb.ft), $\omega_u l_n/14$	**143,706**	**165,231**	**143,706**	**23,105**	**181,147**
M^+.u (lb.ft), $\omega_u l_n/16$	**125,743**	**144,577**	**125,743**	**20,217**	**158,504**
Choose M^+.u	182,899	144,577	182,899	29,407	158,504
M^-.u (lb.ft), $\omega_u l_n/9$	**223,543**	**257,026**	**223,543**	**35,942**	**281,784**
M^-.u (lb.ft), $\omega_u l_n/10$	**201,189**	**231,323**	**201,189**	**32,348**	**253,606**
M^-.u (lb.ft), $\omega_u l_n/11$	**182,899**	**210,294**	**182,899**	**29,407**	**230,551**
M^-.u (lb.ft), $\omega_u l_n/12$	**167,657**	**192,770**	**167,657**	**26,956**	**211,338**
M^-.u (lb.ft), $\omega_u l_n/16$	**125,743**	**144,577**	**125,743**	**20,217**	**158,504**
M^-.u (lb.ft), $\omega_u l_n/24$	**83,829**	**96,385**	**83,829**	**13,478**	**105,669**
	0	**0**	**0**	**0**	**0**
Choose M^-.u	201,189	192,770	201,189	32,348	230,551

Strength Design	76	77	78	CONTINUED		
Flexural Steel Bars (Bottom)	(3) #8	(3) #8	(3) #8			
Flexural Steel Area (in^2), **As**	2.37	2.37	2.37			
Shear Steel Bars	(2) #4	(2) #4	(2) #4			
Shear Steel Area (in^2), **Av**	0.40	0.40	0.40			
spacing of shear steel (in), **s**	12	12	12			
Flexural Steel Bars (Top)	(3) #9	(3) #9	(3) #9			
Flexural Steel Area (in^2), **A's**	3.00	3.00	3.00			
Concrete Strength (psi), f'c	2500	2500	2500			
Flexural Steel (psi), fy	40000	40000	40000			
Shear Steel (psi), fy	40000	40000	40000			
Depth of top comp. block (in), **a**	3.72	3.72	3.72			
ρ	0.00319	0.00319	0.00319			
ρ,min	0.00500	0.00500	0.00500			
Min. reinf. Check	more steel	more steel	more steel			
β_1	0.85	0.85	0.85			
ρ,max = 0.75*ρ,balanced	0.02320	0.02320	0.02320			
Max. reinf. Check	OK	OK	OK			
Depth of bottom comp. block (in), **a**	4.71	4.71	4.71			
ρ' (manual check min. & max.)	0.00403	0.00403	0.00403			
ϕ.b	0.9	0.9	0.9			
ϕ.v	0.85	0.85	0.85			
Bending Strength, M^+.d = ϕM^+.n (lb*ft)	**427,604**	**427,604**	**427,604**			
Check bending strength	OKAY	OKAY	OKAY			
Bending Strength, M^-.d = ϕM^-.n (lb*ft)	**536,824**	**536,824**	**536,824**			
Check bending strength	OKAY	OKAY	OKAY			
Shear Strength, V.d = ϕV.n (lb)	**133,507**	**133,507**	**133,507**			
Check shear strength	OKAY	OKAY	OKAY			
Deflection Design (Valid for simple spans only)						
f.r, modulus of rupture (psi)	375	375	375			
I.g, Gross moment of inertia (in^4)	287,496	287,496	287,496			
y.t, distance from N.A. to tension face	33.00	33.00	33.00			
M.cr, Cracking Moment (lb*ft)	272,250	272,250	272,250			
M.max, Service Moment	150,128	172,286	150,128			
E.s, Elastic Mod. Of Steel (psi)	29,000,000	29,000,000	29,000,000			
E.c, Elastic Mod. Of concrete (psi)	2,850,000	2,850,000	2,850,000			
n = E.s/E.c	10.2	10.2	10.2			
c = d[(nρ*(nρ+2))^1/2-nρ]	13.90	13.90	13.90			
I.cr, Cracked moment of inertia (in^4)	66,537	66,537	66,537			
I.e, Effective moment of intertia (in^4)	**287,496**	**287,496**	**287,496**			
Δ, immediate due to live load (in)	0.002	0.003	0.002			
Span / deflection	**45776**	**39430**	**45776**			
Check live deflection	OKAY	OKAY	OKAY			
Δ, long term from dead load (in)	0.000	0.000	0.000			
Δ, lg. term from sustained live ld. (in)	0.001	0.001	0.001			
Δ, instantaneous live load (in)	0.002	0.002	0.002			
Δ, after attachment of non-structural elements (in.) = Rows 95+96+97	0.003	0.004	0.003			
Span / deflection	**32941**	**28536**	**32941**			
Check live deflection	OKAY	OKAY	OKAY			

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TAN BUILDING 607 A

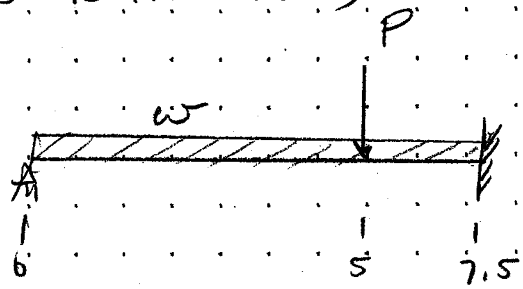
FLOOR ANALYSIS - SUB-AREA 2

TROY 3/10/04

$\boxed{82}$ (83 IS THE SAME)

$h = 66''$, $b = 15''$, $\gamma_c = 150$ PCF

BM. WT. = 1031 PLF



$P_D = \frac{10.5'}{2}(1325\text{ PLF}) = 6956\text{ LB.}$ (REF. 78)

$P_L = \frac{10.5'}{2}(14665\text{ PLF}) = 76991\text{ LB.}$ (REF. 78)

$\omega_D = 1031\text{ PLF} = 1031\text{ PLF}$

$P_u = 1.4P_D + 1.7P_L = 140.6\text{ K}$

$\omega_u = 1.4\omega_D = 1.44\text{ KLF}$

$V_u = \frac{5\omega_u(7.5')}{8} + \frac{P_u(5')}{2(7.5')^3}\left(3(7.5')^2 - (5')^2\right) = \underline{\underline{120\text{ K}}}$

$M_u^+ = \frac{P_u(2.5')^2}{2(7.5')^3}\left(5' + 2(7.5')\right)5' = \underline{\underline{104.1\text{ K}'}}$

$M_u^- = \frac{P_u(2.5')(5')}{2(7.5')^2}(5' + 7.5') = \underline{\underline{195.3\text{ K}'}}$

$\frac{D}{C} = 0.805$

GOVERNED BY SHEAR

LIVE LOAD = 2933 PSF

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TAN BUILDING 607 A

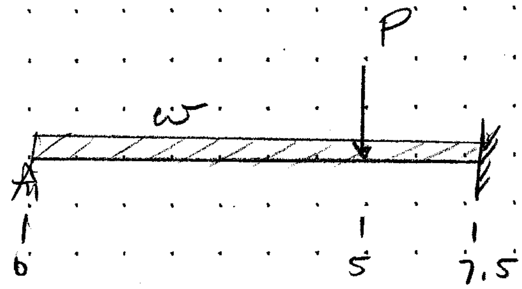
FLOOR ANALYSIS - SUB AREA 2

TROY 3/10/04

84

$h = 66''$, $b = 18''$, $\gamma_c = 150$ PCF

BM. WT. = 1238 PLF



$P_D = \frac{10.5'}{2}(1325 \text{ PLF}) = 6956 \text{ LB.}$ (REF. 76)

$P_L = \frac{10.5'}{2}(14665 \text{ PLF}) = 76991 \text{ LB.}$ (REF. 76)

$\omega_D = 1238 \text{ PLF} = 1238 \text{ PLF}$

$P_u = 1.4 P_D + 1.7 P_L = 140.6 \text{ K}$

$\omega_u = 1.4 \omega_D = 1.73 \text{ KLF}$

$$V_u = \frac{5 \omega_u (7.5')}{8} + \frac{P_u (5')}{2 (7.5')^3} \cdot \left(3(7.5')^2 - (5')^2\right) = \underline{128^K}$$

$$M_u^+ = \frac{P_u (2.5')^2}{2 (7.5')^3}\left(5' + 2(7.5')\right) 5' = \underline{104.1 \text{ K}'}$$

$$M_u^- = \frac{P_u (2.5')(5')}{2 (7.5')^2}\left(5' + 7.5'\right) = \underline{195.3 \text{ K}'}$$

$\frac{D}{C} = 0.776$

GOVERNED BY SHEAR

LIVE LOAD = 2933 PSF

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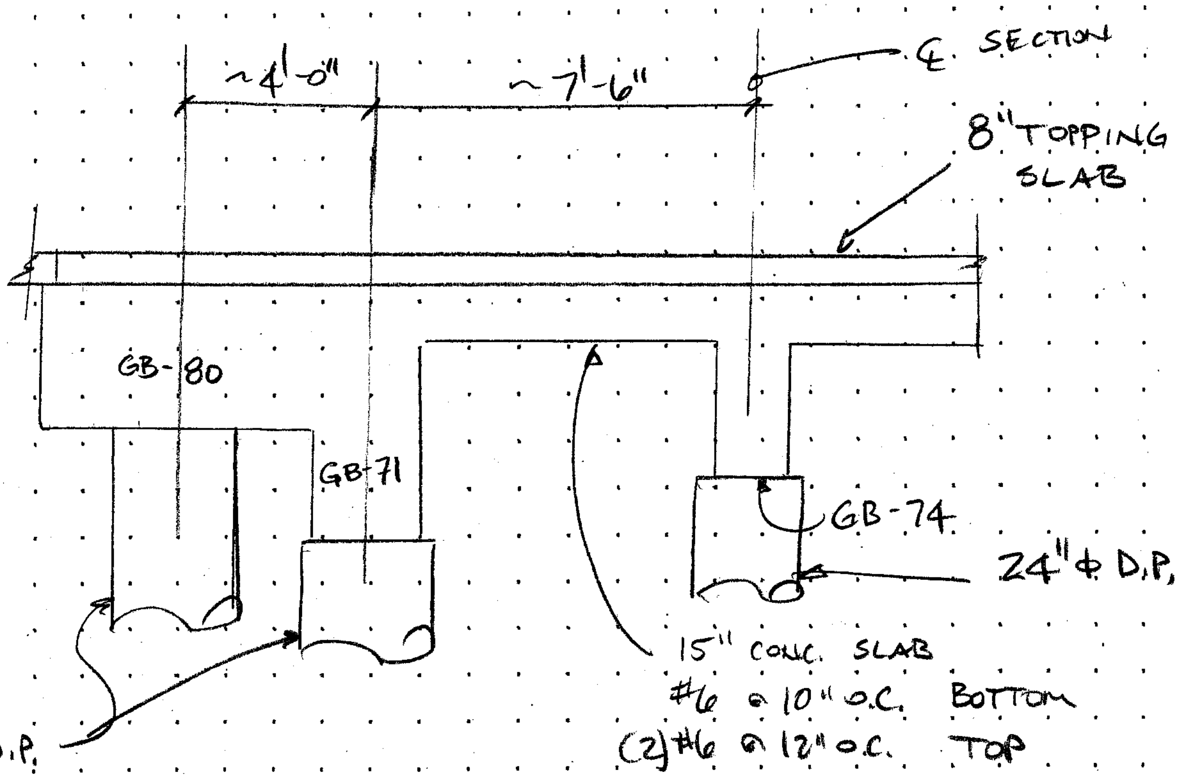
SUB-AREA (4) - REF. B / S115



TYPICAL CROSS SECTION
NO SCALE

CAPACITY = 1542 PLF (GOVERNED BY 73)
LIVE LOAD

C.I.P. Concrete Beam or Slab Analysis - Determine max. superimposed (live) load allowed

Beam Label	Typical 15" slab	71	72	73	74
Depth of Beam (in) **h**	15	66	66	48	48
Depth to Reinf. (in) **d**	12	62	62	44	44
Width of Beam (in) **b**	12	26	26	18	18
Slab Section or Beam Size	**12 x 15**	**26 x 66**	**26 x 66**	**18 x 48**	**18 x 48**
Design Criteria					
	360	360	360	360	360
Δ limit due to Long-Term Loads (L /) applied after non-structural elements are attached	480	480	480	480	480
% of live load that is long-term	20%	20%	20%	20%	20%
% of live load that is not long-term	80%	80%	80%	80%	80%
$\lambda = \xi/(1+50p')$, $\xi = 2.0$ for long-term l	1.53	2.00	2.00	2.00	2.00
Concrete unit weight (pcf)	150	150	150	150	150
Floor Uniform Dead Load (psf)	100	288	288	288	288
Floor Uniform Live Load (psf)	**2380**	**4694**	**5000**	**1542**	**1824**
Floor Beam Linear Dead Load (plf)	187.50	1787.50	1787.50	900.00	900.00
Analysis, ref. ACI 318-99, sections 8.7 (span length), 8.3 (methods of analysis), and 11.1.3.1					
Span (Ctr to Ctr of Supports) (ft)	7.5	10.5	7.75	10.5	10.5
Width of Supports (in)	24	28	28	24	24
Analyze Ctr-Ctr(0) or Clr Span(1)	1	1	1	1	1
Effective Span (ft)	5.5	8.16666667	5.41666667	8.5	8.5
Tributary width (ft)	1.00	5.75	5.75	7.50	7.50
Include beam wt? No(0)/Yes(1)	1	1	1	1	1
Uniform Dead Load (plf)	287.50	3443.50	3443.50	3060.00	3060.00
Uniform Live Load (plf)	2380.00	26990.50	28750.00	11565.00	13680.00
U = 1.4D + 1.7L (plf)	4449	50705	53696	23945	27540
V.u (lb), $1.15\omega_u \ell_n/2$	**14,068**	**238,101**	**167,240**	**117,029**	**134,602**
V.u (lb), $\omega_u \ell_n/2$	**12,233**	**207,044**	**145,426**	**101,764**	**117,045**
V.u (lb), $\omega_u \ell_n/2 - d\omega_u$	**7,785**	**-54,930**	**-132,002**	**13,968**	**16,065**
Choose V.u	12,233	207,044	167,240	117,029	117,045
M^+.u (lb.ft), $\omega_u \ell_n/8$	**16,821**	**422,716**	**196,932**	**216,249**	**248,721**
M^+.u (lb.ft), $\omega_u \ell_n/11$	**12,233**	**307,430**	**143,223**	**157,272**	**180,888**
M^+.u (lb.ft), $\omega_u \ell_n/14$	**9,612**	**241,552**	**112,532**	**123,571**	**142,126**
M^+.u (lb.ft), $\omega_u \ell_n/16$	**8,410**	**211,358**	**98,466**	**108,124**	**124,360**
Choose M^+.u	8,410	211,358	143,223	157,272	124,360
M^-.u (lb.ft), $\omega_u \ell_n/9$	**14,952**	**375,747**	**175,050**	**192,221**	**221,085**
M^-.u (lb.ft), $\omega_u \ell_n/10$	**13,457**	**338,173**	**157,545**	**172,999**	**198,977**
M^-.u (lb.ft), $\omega_u \ell_n/11$	**12,233**	**307,430**	**143,223**	**157,272**	**180,888**
M^-.u (lb.ft), $\omega_u \ell_n/12$	**11,214**	**281,810**	**131,288**	**144,166**	**165,814**
M^-.u (lb.ft), $\omega_u \ell_n/16$	**8,410**	**211,358**	**98,466**	**108,124**	**124,360**
M^-.u (lb.ft), $\omega_u \ell_n/24$	**5,607**	**140,905**	**65,644**	**72,083**	**82,907**
	0	**0**	**0**	**0**	**0**
Choose M^-.u	12,233	307,430	175,050	192,221	180,888

Strength Design	15" slab	71	72	73	74
Flexural Steel Bars (Bottom)	(1) #6				
Flexural Steel Area (in^2), **As**	0.44				
Shear Steel Bars	None				
Shear Steel Area (in^2), **Av**	0.00				
spacing of shear steel (in), **s**	999				
Flexural Steel Bars (Top)	(2) #6				
Flexural Steel Area (in^2), **A's**	0.88				
Concrete Strength (psi), f'c	2500				
Flexural Steel (psi), fy	40000				
Shear Steel (psi), fy	40000				
Depth of top comp. block (in), **a**	0.69				
ρ	0.00306				
ρ,min	0.00500				
Min. reinf. Check	more steel				
β_1	0.85				
ρ,max = 0.75*ρ,balanced	0.02320				
Max. reinf. Check	OK				
Depth of bottom comp. block (in), **a**	1.38				
ρ' (manual check min. & max.)	0.00611				
ϕ.b	0.9				
ϕ.v	0.85				
Bending Strength, M^+.d = ϕM^+.n (lb*ft)	**15,384**				
Check bending strength	OKAY				
Bending Strength, M^-.d = ϕM^-.n (lb*ft)	**29,858**				
Check bending strength	OKAY				
Shear Strength, V.d = ϕV.n (lb)	**12,240**				
Check shear strength	OKAY				
Deflection Design (Valid for simple spans only)					
f.r, modulus of rupture (psi)	375				
I.g, Gross moment of inertia (in^4)	3,375				
y.t, distance from N.A. to tension face	7.50				
M.cr, Cracking Moment (lb*ft)	14,063				
M.max, Service Moment	10,086				
E.s, Elastic Mod. Of Steel (psi)	29,000,000				
E.c, Elastic Mod. Of concrete (psi)	2,850,000				
n = E.s/E.c	10.2				
c = d[(nρ*(nρ+2))^1/2-nρ]	2.64				
I.cr, Cracked moment of inertia (in^4)	466				
I.e, Effective moment of intertia (in^4)	**3,375**				
Δ, immediate due to live load (in)	0.005				
Span / deflection	**12955**				
Check live deflection	OKAY				
Δ, long term from dead load (in)	0.000				
Δ, lg. term from sustained live ld. (in)	0.003				
Δ, instantaneous live load (in)	0.004				
Δ, after attachment of non-structural elements (in.) = Rows 95+96+97	0.007				
Span / deflection	**9451**				
Check live deflection	OKAY				

C.I.P. Concrete Beam or Slab Analysis - Determine max. superimposed (live) load allowed

Beam Label	**75**	**None**	**81**	**79**	**80**
Depth of Beam (in) **h**	48	0	36	36	36
Depth to Reinf. (in) **d**	44	0	32	32	32
Width of Beam (in) **b**	18	0	66	66	66
Slab Section or Beam Size	**18 x 48**	**0 x 0**	**66 x 36**	**66 x 36**	**66 x 36**
Design Criteria					
	360	360	360	360	360
Δ limit due to Long-Term Loads (L /) applied after non-structural elements are attached	480	480	480	480	480
% of live load that is long-term	**20%**	**20%**	**20%**	**20%**	**20%**
% of live load that is not long-term	80%	80%	80%	80%	80%
$\lambda = \xi/(1+50\rho')$, ξ = 2.0 for long-term	1.63	2.00	2.00	2.00	2.00
Concrete unit weight (pcf)	150	150	150	150	150
Floor Uniform Dead Load (psf)	**288**	**0**	**100**	**100**	**100**
Floor Uniform Live Load (psf)	**2440**	**0**	**5000**	**5000**	**5000**
Floor Beam Linear Dead Load (plf)	900.00	0.00	2475.00	2475.00	2475.00
Analysis, ref. ACI 318-99, sections 8.7 (span length), 8.3 (methods of analysis), and 11.1.3.1					
Span (Ctr to Ctr of Supports) (ft)	7.75	0	10.5	5.25	10.5
Width of Supports (in)	24	0	28	28	28
Analyze Ctr-Ctr(0) or Clr Span(1)	1	0	1	1	1
Effective Span (ft)	5.75	0	8.16666667	2.91666667	8.16666667
Tributary width (ft)	7.50	0.00	4.00	4.00	4.00
Include beam wt? No(0)/Yes(1)	1	0	1	1	1
Uniform Dead Load (plf)	3060.00	0.00	2875.00	2875.00	2875.00
Uniform Live Load (plf)	18300.00	0.00	20000.00	20000.00	20000.00
U = 1.4D + 1.7L (plf)	35394	0	38025	38025	38025
V.u (lb), $1.15\omega_u\ell_n/2$	**117,021**	**0**	**178,559**	**63,771**	**178,559**
V.u (lb), $\omega_u\ell_n/2$	**101,758**	**0**	**155,269**	**55,453**	**155,269**
V.u (lb), $\omega_u\ell_n/2 - d\omega_u$	**-28,020**	**0**	**53,869**	**-45,947**	**53,869**
Choose V.u	117,021	0	178,559	63,771	155,269
M^+.u (lb.ft), $\omega_u\ell_n/8$	**146,277**	**0**	**317,007**	**40,435**	**317,007**
M^+.u (lb.ft), $\omega_u\ell_n/11$	**106,383**	**0**	**230,551**	**29,407**	**230,551**
M^+.u (lb.ft), $\omega_u\ell_n/14$	**83,587**	**0**	**181,147**	**23,105**	**181,147**
M^+.u (lb.ft), $\omega_u\ell_n/16$	**73,138**	**0**	**158,504**	**20,217**	**158,504**
Choose M^+.u	106,383	0	230,551	29,407	158,504
M^-.u (lb.ft), $\omega_u\ell_n/9$	**130,024**	**0**	**281,784**	**35,942**	**281,784**
M^-.u (lb.ft), $\omega_u\ell_n/10$	**117,021**	**0**	**253,606**	**32,348**	**253,606**
M^-.u (lb.ft), $\omega_u\ell_n/11$	**106,383**	**0**	**230,551**	**29,407**	**230,551**
M^-.u (lb.ft), $\omega_u\ell_n/12$	**97,518**	**0**	**211,338**	**26,956**	**211,338**
M^-.u (lb.ft), $\omega_u\ell_n/16$	**73,138**	**0**	**158,504**	**20,217**	**158,504**
M^-.u (lb.ft), $\omega_u\ell_n/24$	**48,759**	**0**	**105,669**	**13,478**	**105,669**
	0	**0**	**0**	**0**	**0**
Choose M^-.u	130,024	0	281,784	32,348	230,551

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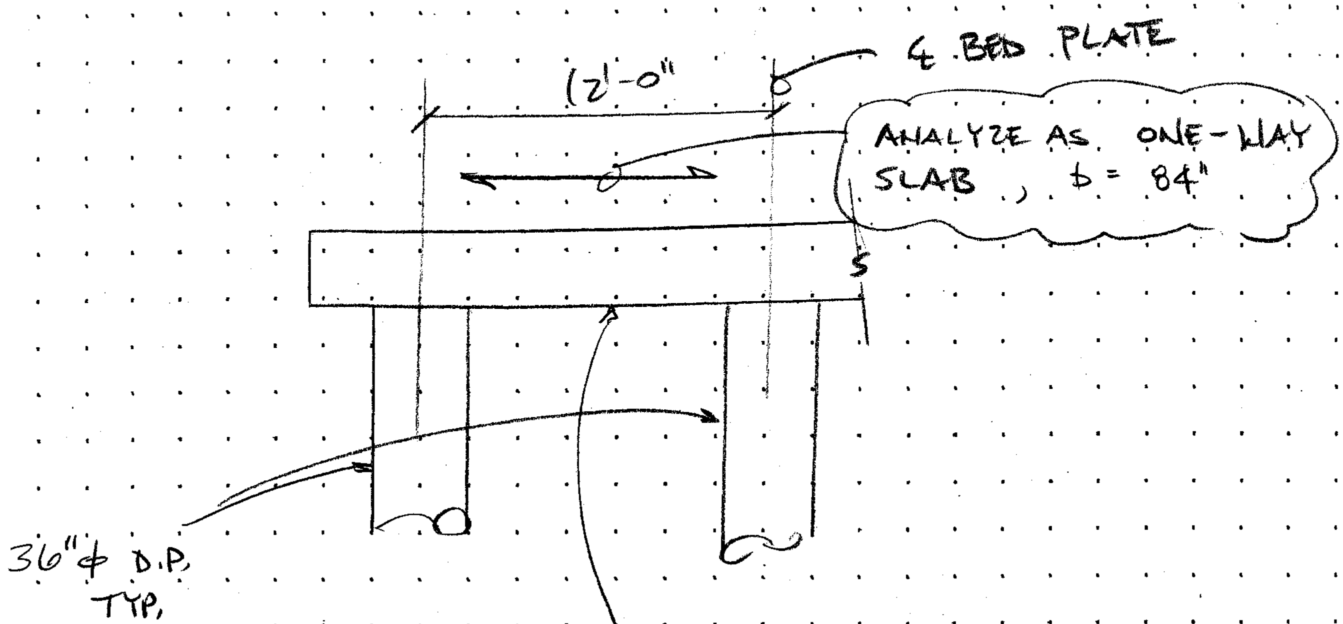
SUB - AREA 5 REF. D / S115



NOTE: IN A 7'-0" WIDE SECTION OF SLAB, THERE ARE AT LEAST (9) #8 TOP BARS AND (8) #8 BOTTOM BARS

24" DEEP SLAB
W/ #8 @ 9" o/c TOP BARS
#8 @ 10" o/c BOTTOM BARS

d to A_s = 20"
d to A'_s = 22"

ALSO CHECK PERPENDICULAR DIRECTION
#6 @ 6" o/c TOP, #6 @ 12" o/c BOTTOM

CAPACITY = 1490 PSF LINE LOAD

C.I.P. Concrete Beam or Slab Analysis - Determine max. superimposed (live) load allowed

Beam Label	12-ft span 24" slab	7-span 24" slab	None	None	None
Depth of Beam (in) **h**	24	24	0	0	0
Depth to Reinf. (in) **d**	20	20	0	0	0
Width of Beam (in) **b**	84	12	0	0	0
Slab Section or Beam Size	84 x 24	12 x 24	0 x 0	0 x 0	0 x 0
Design Criteria					
	360	360	360	360	360
Δ limit due to Long-Term Loads (L /) applied after non-structural elements are attached	480	480	480	480	480
% of live load that is long-term	20%	20%	20%	20%	20%
% of live load that is not long-term	80%	80%	80%	80%	80%
$\lambda = \xi/(1+50p')$, $\xi = 2.0$ for long-term lo	1.65	1.69	2.00	2.00	2.00
Concrete unit weight (pcf)	150	150	150	150	150
Floor Uniform Dead Load (psf)	0	0	0	0	0
Floor Uniform Live Load (psf)	1490	2734	0	0	0
Floor Beam Linear Dead Load (plf)	2100.00	300.00	0.00	0.00	0.00
Analysis, ref. ACI 318-99, sections 8.7 (span length), 8.3 (methods of analysis), and 11.1.3.1					
Span (Ctr to Ctr of Supports) (ft)	12	7	0	0	0
Width of Supports (in)	36	36	0	0	0
Analyze Ctr-Ctr(0) or Clr Span(1)	0	0	0	0	0
Effective Span (ft)	12	7	0	0	0
Tributary width (ft)	7.00	1.00	0.00	0.00	0.00
Include beam wt? No(0)/Yes(1)	1	1	0	0	0
Uniform Dead Load (plf)	2100.00	300.00	0.00	0.00	0.00
Uniform Live Load (plf)	10430.00	2734.00	0.00	0.00	0.00
U = 1.4D + 1.7L (plf)	20671	5068	0	0	0
V.u (lb), $1.15\omega_u l_n/2$	**142,630**	**20,398**	**0**	**0**	**0**
V.u (lb), $\omega_u l_n/2$	**124,026**	**17,737**	**0**	**0**	**0**
V.u (lb), $\omega_u l_n/2 - d\omega_u$	**89,574**	**9,291**	**0**	**0**	**0**
Choose V.u	142,630	20,398	0	0	0
M^+.u (lb.ft), $\omega_u l_n/8$	**372,078**	**31,040**	**0**	**0**	**0**
M^+.u (lb.ft), $\omega_u l_n/11$	**270,602**	**22,575**	**0**	**0**	**0**
M^+.u (lb.ft), $\omega_u l_n/14$	**212,616**	**17,737**	**0**	**0**	**0**
M^+.u (lb.ft), $\omega_u l_n/16$	**186,039**	**15,520**	**0**	**0**	**0**
Choose M^+.u	270,602	22,575	0	0	0
M^-.u (lb.ft), $\omega_u l_n/9$	**330,736**	**27,591**	**0**	**0**	**0**
M^-.u (lb.ft), $\omega_u l_n/10$	**297,662**	**24,832**	**0**	**0**	**0**
M^-.u (lb.ft), $\omega_u l_n/11$	**270,602**	**22,575**	**0**	**0**	**0**
M^-.u (lb.ft), $\omega_u l_n/12$	**248,052**	**20,694**	**0**	**0**	**0**
M^-.u (lb.ft), $\omega_u l_n/16$	**186,039**	**15,520**	**0**	**0**	**0**
M^-.u (lb.ft), $\omega_u l_n/24$	**124,026**	**10,347**	**0**	**0**	**0**
	0	0	0	0	0
Choose M^-.u	330,736	27,591	0	0	0

Strength Design	24" slab	24" slab	None	None	None
Flexural Steel Bars (Bottom)	(8) #8	(1) #6			
Flexural Steel Area (in^2), **As**	6.32	0.44			
Shear Steel Bars	None	None			
Shear Steel Area (in^2), **Av**	0.00	0.00			
spacing of shear steel (in), **s**	999	999			
Flexural Steel Bars (Top)	(9) #8	(2) #6			
Flexural Steel Area (in^2), **A's**	7.11	0.88			
Concrete Strength (psi), f'c	2500	2500			
Flexural Steel (psi), fy	40000	40000			
Shear Steel (psi), fy	40000	40000			
Depth of top comp. block (in), **a**	1.42	0.69			
ρ	0.00376	0.00183			
ρ,min	0.00500	0.00500			
Min. reinf. Check	more steel	more steel			
β_1	0.85	0.85			
ρ,max = 0.75*ρ,balanced	0.02320	0.02320			
Max. reinf. Check	OK	OK			
Depth of bottom comp. block (in), **a**	1.59	1.38			
ρ' (manual check min. & max.)	0.00423	0.00367			
ϕ.b	0.9	0.9			
ϕ.v	0.85	0.85			
Bending Strength, $M^+.d = \phi M^+.n$ (lb*ft)	**365,774**	**25,944**			
Check bending strength	OKAY	OKAY			
Bending Strength, $M^-.d = \phi M^-.n$ (lb*ft)	**409,608**	**50,978**			
Check bending strength	OKAY	OKAY			
Shear Strength, V.d = ϕV.n (lb)	**142,800**	**20,400**			
Check shear strength	OKAY	OKAY			
Deflection Design (Valid for simple spans only)					
f.r, modulus of rupture (psi)	375	375			
I.g, Gross moment of inertia (in^4)	96,768	13,824			
y.t, distance from N.A. to tension face	12.00	12.00			
M.cr, Cracking Moment (lb*ft)	252,000	36,000			
M.max, Service Moment	225,540	18,583			
E.s, Elastic Mod. Of Steel (psi)	29,000,000	29,000,000			
E.c, Elastic Mod. Of concrete (psi)	2,850,000	2,850,000			
n = E.s/E.c	10.2	10.2			
c = d[(nρ*(nρ+2))^1/2-nρ]	4.82	3.51			
I.cr, Cracked moment of inertia (in^4)	17,954	1,390			
I.e, Effective moment of intertia (in^4)	**96,768**	**13,824**			
Δ, immediate due to live load (in)	0.018	0.004			
Span / deflection	**8161**	**22407**			
Check live deflection	OKAY	OKAY			
Δ, long term from dead load (in)	0.000	0.000			
Δ, lg. term from sustained live ld. (in)	0.009	0.002			
Δ, instantaneous live load (in)	0.014	0.003			
Δ, after attachment of non-structural elements (in.) = Rows 95+96+97	0.023	0.005			
Span / deflection	**6135**	**16746**			
Check live deflection	OKAY	OKAY			

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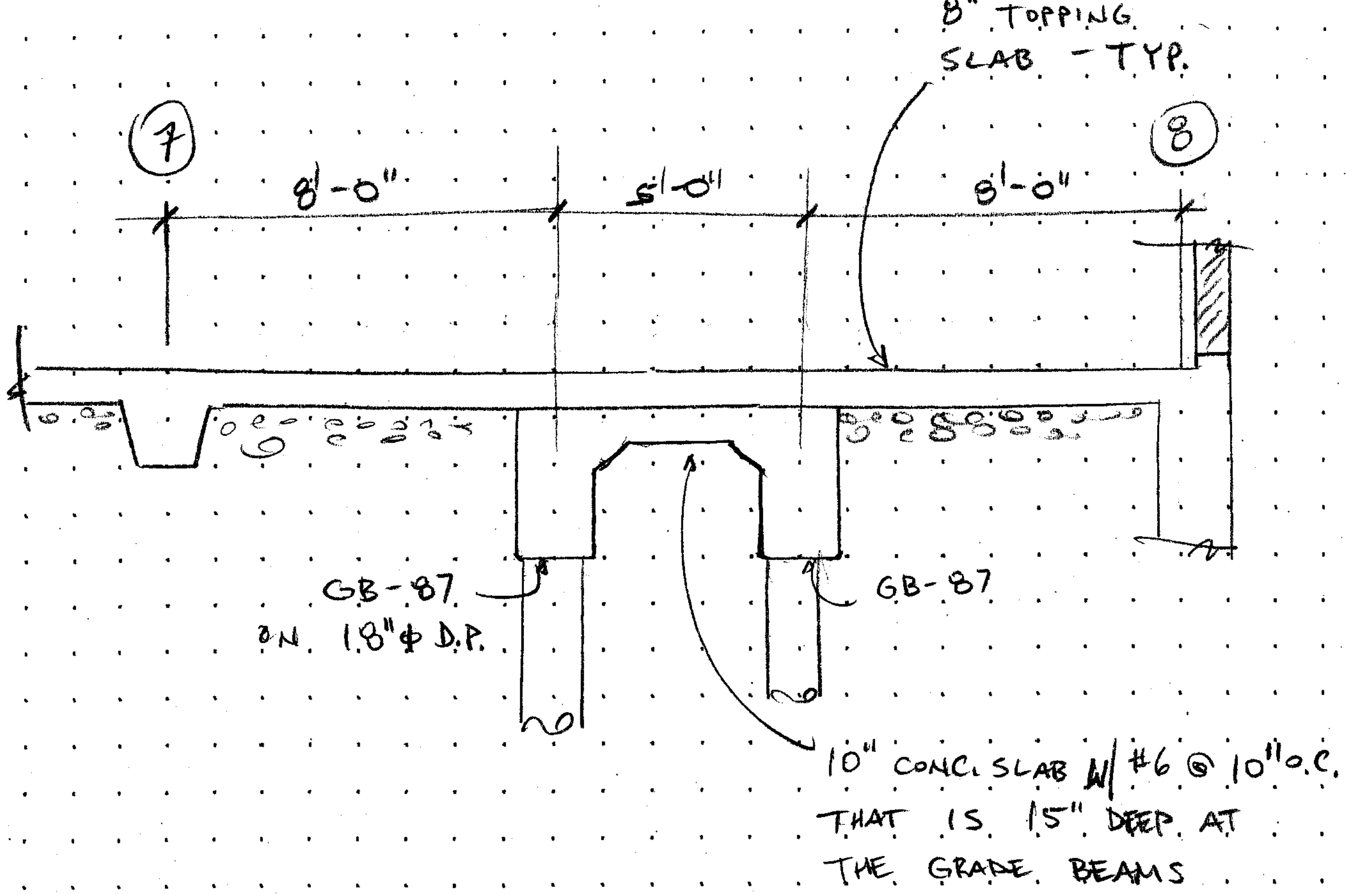
SUB-AREA 7 REF. E / S115



d to A_s = 7"
d to A'_s = 13"

* CAPACITY = 500 PSF (SIM. TO SUB-AREA 1) LIVE LOAD

* EXCEPT AT THE TRACKS WHERE AN 1895 PSF SUPERIMPOSED LIVE LOAD IS ALLOWED

C.I.P. Concrete Beam or Slab Analysis - Determine max. superimposed (live) load allowed

Beam Label	Typical 15" slab	86	87	None	None
Depth of Beam (in) **h**	10	35	35	0	0
Depth to Reinf. (in) **d**	7	31	31	0	0
Width of Beam (in) **b**	10	20	20	0	0
Slab Section or Beam Size	**10 x 10**	**20 x 35**	**20 x 35**	**0 x 0**	**0 x 0**
Design Criteria					
	360	360	360	360	360
Δ limit due to Long-Term Loads (L /) applied after non-structural elements are attached	480	480	480	480	480
% of live load that is long-term	20%	20%	20%	20%	20%
% of live load that is not long-term	80%	80%	80%	80%	80%
$\lambda = \xi/(1+50p')$, $\xi = 2.0$ for long-term lo	1.52	2.00	2.00	2.00	2.00
Concrete unit weight (pcf)	150	150	150	150	150
Floor Uniform Dead Load (psf)	100	288	288	0	0
Floor Uniform Live Load (psf)	2335	1990	1895	0	0
Floor Beam Linear Dead Load (plf)	104.17	729.17	729.17	0.00	0.00
Analysis, ref. ACI 318-99, sections 8.7 (span length), 8.3 (methods of analysis), and 11.1.3.1					
Span (Ctr to Ctr of Supports) (ft)	5	10.33	9.5	0	0
Width of Supports (in)	20	18	18	0	0
Analyze Ctr-Ctr(0) or Clr Span(1)	1	1	1	0	0
Effective Span (ft)	3.33333333	8.83	8	0	0
Tributary width (ft)	0.83	5.00	5.00	0.00	0.00
Include beam wt? No(0)/Yes(1)	1	1	1	0	0
Uniform Dead Load (plf)	187.47	2169.17	2169.17	0.00	0.00
Uniform Live Load (plf)	1945.06	9950.00	9475.00	0.00	0.00
U = 1.4D + 1.7L (plf)	3569	19952	19144	0	0
V.u (lb), $1.15\omega_u l_n/2$	**6,841**	**101,300**	**88,064**	**0**	**0**
V.u (lb), $\omega_u l_n/2$	**5,948**	**88,087**	**76,577**	**0**	**0**
V.u (lb), $\omega_u l_n/2 - d\omega_u$	**3,866**	**36,545**	**27,121**	**0**	**0**
Choose V.u	5,948	88,087	88,064	0	0
M^+.u (lb.ft), $\omega_u l_n/8$	**4,957**	**194,453**	**153,155**	**0**	**0**
M^+.u (lb.ft), $\omega_u l_n/11$	**3,605**	**141,420**	**111,385**	**0**	**0**
M^+.u (lb.ft), $\omega_u l_n/14$	**2,833**	**111,116**	**87,517**	**0**	**0**
M^+.u (lb.ft), $\omega_u l_n/16$	**2,479**	**97,226**	**76,577**	**0**	**0**
Choose M^+.u	2,479	97,226	111,385	0	0
M^-.u (lb.ft), $\omega_u l_n/9$	**4,406**	**172,847**	**136,137**	**0**	**0**
M^-.u (lb.ft), $\omega_u l_n/10$	**3,966**	**155,562**	**122,524**	**0**	**0**
M^-.u (lb.ft), $\omega_u l_n/11$	**3,605**	**141,420**	**111,385**	**0**	**0**
M^-.u (lb.ft), $\omega_u l_n/12$	**3,305**	**129,635**	**102,103**	**0**	**0**
M^-.u (lb.ft), $\omega_u l_n/16$	**2,479**	**97,226**	**76,577**	**0**	**0**
M^-.u (lb.ft), $\omega_u l_n/24$	**1,652**	**64,818**	**51,052**	**0**	**0**
	0	**0**	**0**	**0**	**0**
Choose M^-.u	3,605	141,420	136,137	0	0

Strength Design	15" slab	86	87	None	None
Flexural Steel Bars (Bottom)	(1) #6				
Flexural Steel Area (in^2), **As**	0.44				
Shear Steel Bars	None				
Shear Steel Area (in^2), **Av**	0.00				
spacing of shear steel (in), **s**	999				
Flexural Steel Bars (Top)	(1) #6				
Flexural Steel Area (in^2), **A's**	0.44				
Concrete Strength (psi), f'c	2500				
Flexural Steel (psi), fy	40000				
Shear Steel (psi), fy	40000				
Depth of top comp. block (in), **a**	0.83				
ρ	0.00629				
ρ,min	0.00500				
Min. reinf. Check	OK				
β_1	0.85				
ρ,max = 0.75*ρ,balanced	0.02320				
Max. reinf. Check	OK				
Depth of bottom comp. block (in), **a**	0.83				
ρ' (manual check min. & max.)	0.00629				
ϕ.b	0.9				
ϕ.v	0.85				
Bending Strength, M^+.d = ϕM^+.n (lb*ft)	**8,693**				
Check bending strength	OKAY				
Bending Strength, M^-.d = ϕM^-.n (lb*ft)	**8,693**				
Check bending strength	OKAY				
Shear Strength, V.d = ϕV.n (lb)	**5,950**				
Check shear strength	OKAY				
Deflection Design (Valid for simple spans only)					
f.r, modulus of rupture (psi)	375				
I.g, Gross moment of inertia (in^4)	833				
y.t, distance from N.A. to tension face	5.00				
M.cr, Cracking Moment (lb*ft)	5,208				
M.max, Service Moment	2,962				
E.s, Elastic Mod. Of Steel (psi)	29,000,000				
E.c, Elastic Mod. Of concrete (psi)	2,850,000				
n = E.s/E.c	10.2				
c = d[(nρ*(nρ+2))^1/2-nρ]	2.10				
I.cr, Cracked moment of inertia (in^4)	138				
I.e, Effective moment of intertia (in^4)	**833**				
Δ, immediate due to live load (in)	0.002				
Span / deflection	**17583**				
Check live deflection	OKAY				
Δ, long term from dead load (in)	0.000				
Δ, lg. term from sustained live ld. (in)	0.001				
Δ, instantaneous live load (in)	0.002				
Δ, after attachment of non-structural elements (in.) = Rows 95+96+97	0.003				
Span / deflection	**12839**				
Check live deflection	OKAY				

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Clarification Item #1

Date: June 8, 2004

Project: Idaho National Labs
TAN Bldg. 607A
Transporter & Tank Support
Idaho Falls, Idaho

To: Jeff Towers
Portage Environmental
1075 South Utah St., Ste 200
Idaho Falls, ID 83402

From: Troy Leistiko, P.E.

Re: Wood Confinement Plate

Reference our previous letter dated April 30th, 2004.

Item 1: In order to provide a bolted connection embedded less than 2-inches into the existing concrete floor, the wood confinement plates shall be fastened to the concrete floor with 3/8-inch diameter 'Hilti' Kwik Bolts as described on the attached DETAIL A and DETAIL B.

END OF CLARIFICATION ITEM #1

Attachments: DETAIL A, DETAIL B

PROFESSIONAL ENGINEER
REGISTERED
9081
6/8/04
STATE OF IDAHO
TROY LEISTIKO

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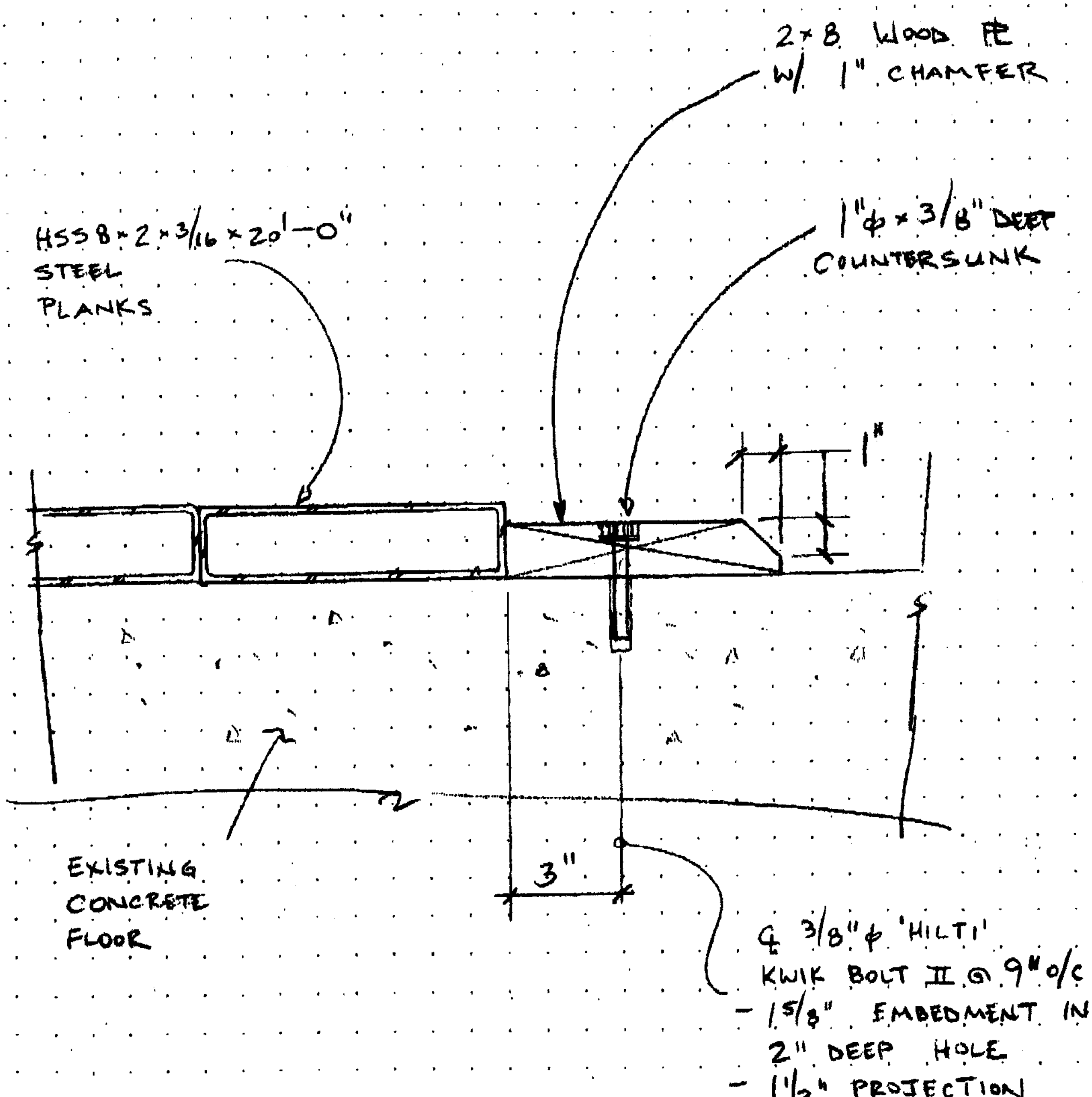
TAN BLDG. 607A
TRANSPORTER + TANK SUPPORT
CLARIFICATION ITEM #1

1/2

DATE: 6/8/04

DESIGN BY: TROY



A WOOD CONFINEMENT PLATE
SCALE: 3" = 1'-0"

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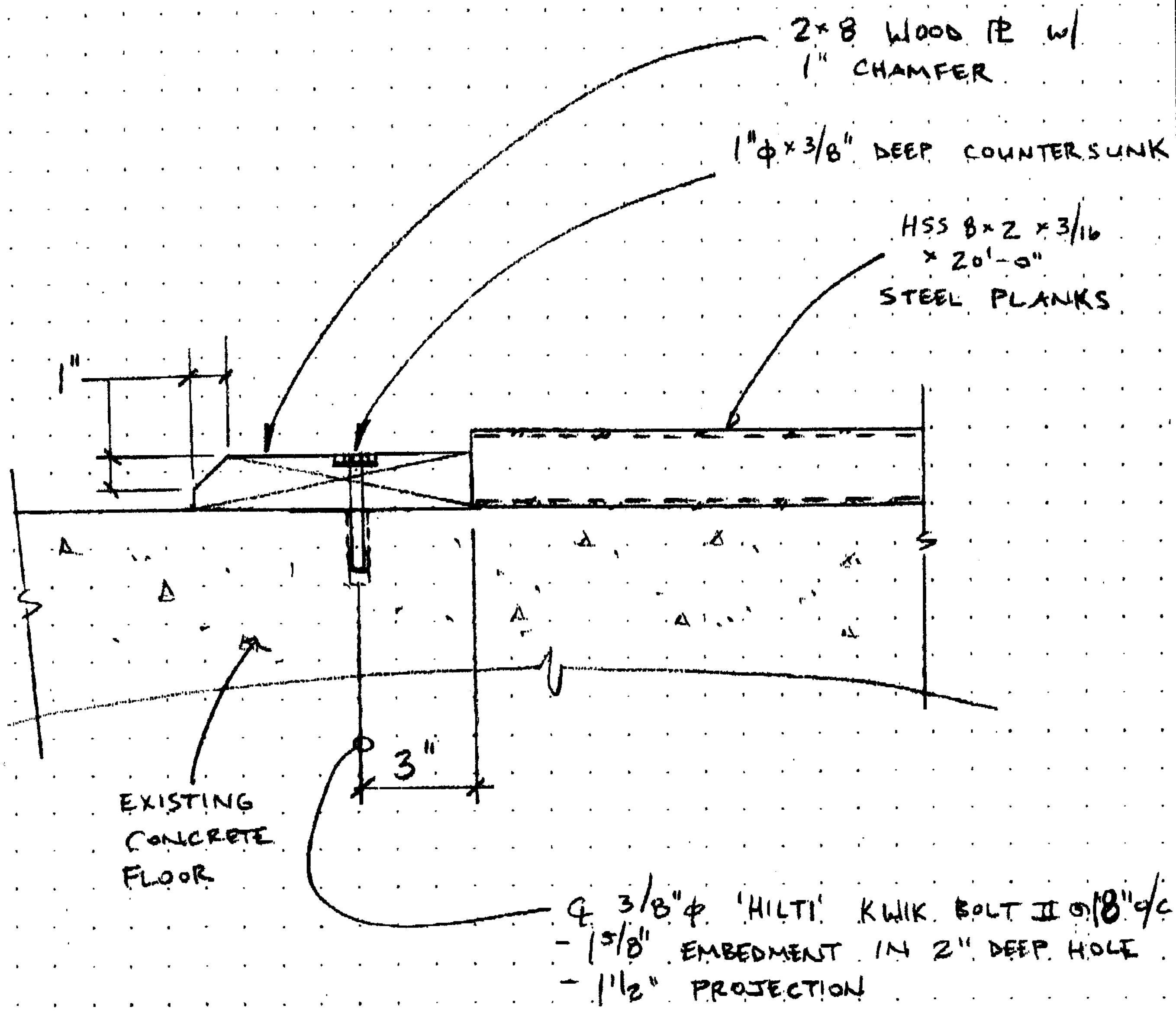
TAN BLDG. 607A
TRANSPORTER + TANK SUPPORT
CLARIFICATION ITEM #1

2/2

DATE: 6/8/04

DESIGN BY: TROY

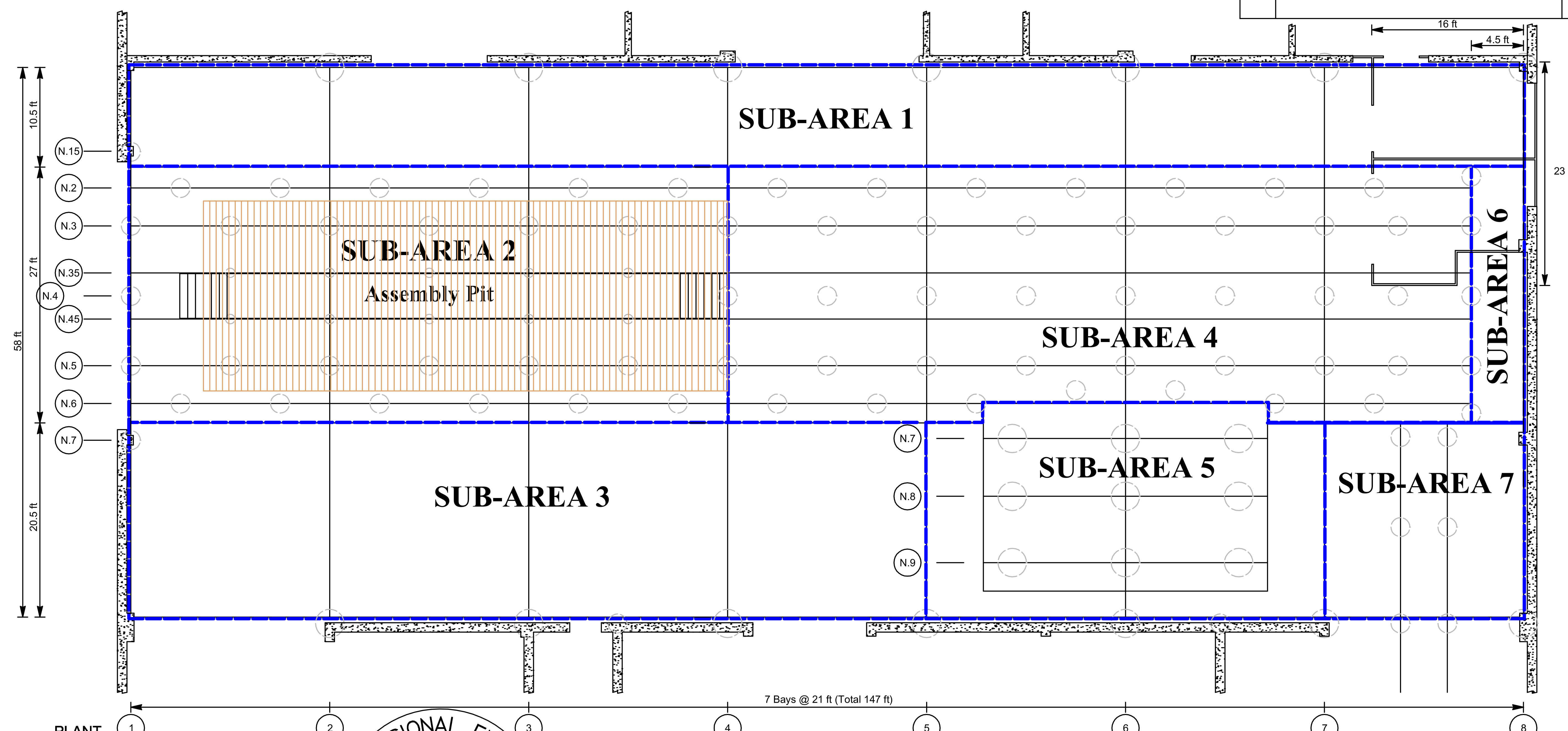


(B) WOOD CONFINEMENT PLATE

SCALE: 3" = 1'—0"

Attachment 2

Drawing No. P-FFA/CO-PM2A-003



REVISIONS		
REV	DESCRIPTION	EFFECTIVE DATE

PROFESSIONAL ENGINEER
REGISTERED
10393
STATE OF IDAHO
BRADY J. ORCHARD

REVISION: 0
ORIGINAL SIGNED BY: Brady J. Orchard, PE
SEAL NUMBER: 10393
DATE ORIGINAL SIGNED: 05/12/04
ORIGINAL STORED AT: Portage Environmental, Inc.; Idaho Falls, ID

LEGEND

BUILDINGS AND STRUCTURES
SUB-AREA BOUNDARY
DRILLED PIERS

FLOOR LOADING CAPACITIES

SUB-AREA 1: 500 POUNDS PER SQUARE FOOT.
SUB-AREA 2: 2615 POUNDS PER SQUARE FOOT, EXCEPT THE ASSEMBLY PIT WHICH IS LIMITED TO 285 POUNDS PER SQUARE FOOT.
SUB-AREA 3: 500 POUNDS PER SQUARE FOOT.
SUB-AREA 4: 1542 POUNDS PER SQUARE FOOT.
SUB-AREA 5: 1490 POUNDS PER SQUARE FOOT.
SUB-AREA 6: 500 POUNDS PER SQUARE FOOT.
SUB-AREA 7: 500 POUNDS PER SQUARE FOOT, EXCEPT THE 6'2" WIDE STRIP THAT SUPPORTS THE RAILROAD TRACKS WHICH CAN SUPPORT UP TO 1895 POUNDS PER SQUARE FOOT (ECLIPSE 2004).

NOTE: FLOOR LOADING CAPACITIES BASED ON EVALUATION OF ORIGINAL CONSTRUCTION DRAWINGS. FLOOR LOADING DOES NOT ACCOUNT FOR ANY SUBSEQUENT MODIFICATIONS.

Portage ENVIRONMENTAL
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IDAHO NATIONAL ENGINEERING AND ENVIRONMENTAL LABORATORY
TEST AREA NORTH/TECHNICAL SUPPORT FACILITY
PM-2A REMEDIATION PROJECT
HIGH BAY ASSEMBLY SHOP CONCRETE FLOOR LOADING CAPACITY

SHEET:	1 OF 1		SCALE:	SHOWN
DESIGN:	ECLIPSE	03/11/04	QA LEVEL: Q	DWG SIZE: B
DRAWN:	M.P. LUNDELL	04/27/04	CONTRACT NO.	2073.00
CHECKED:	J. TOWERS	05/12/04	DRAWING NUMBER: P-FFA/CO-PM2A-003	REV 0
APPROVED:	B.J. ORCHARD	05/12/04		